NAVAL FACILITIES ENGINEERING COMMAND
200 Stovall Street
Alexandria, Virginia 22332-2300

Approved for Public Release

NOTICE OF INACTIVE
FOR NEW DESIGN

DM 26.2
NOTICE 1
1 May 1996

COASTAL PROTECTION

Design Manual 26.2, dated April 1982, is inactive for new design and shall no longer be used as guidance reference for new design.

New designs should use COE Shore Protection Manual, 1984. CAUTION: The supersession information is valid as of the date of this notice and may be superseded by subsequent revisions of the superseding document.

Application for copies of Shore Protection Manual should be addressed to:

U.S. Army Corps of Engineers Publications Depot
2803 52nd Avenue
Hyattsville, MD 20781-1102
Telephone: (301) 436-2063

DESIGN MANUAL 26.02
APRIL 1982

Design Manual 26.02
April 1982
Notice 1
1 May 1996
CCB #37 Third Quarter 1996

CCB Application Notes:

1. Character(s) preceded & followed by these symbols (À Ù) or (Ú ¿) are super- or subscripted, respectively.
   EXAMPLES: 42mÀ3Ù = 42 cubic meters
   COÚ2¿ = carbon dioxide

2. All degree symbols have been replaced with the word deg.

3. All plus or minus symbols have been replaced with the symbol +/-.

4. All table note letters and numbers have been enclosed in square brackets in both the table and below the table.

5. Whenever possible, mathematical symbols have been replaced with their proper name and enclosed in square brackets.

COASTAL PROTECTION

DESIGN MANUAL 26.2

APPROVED FOR PUBLIC RELEASE

DEPARTMENT OF THE NAVY
NAVAL FACILITIES ENGINEERING COMMAND
200 STOVALL STREET
ALEXANDRIA, VIRGINIA 22332

ABSTRACT

Design criteria and planning guidelines for qualified engineers are presented for design of coastal structures. Section 1 is a presentation of applicable wave theory and wave transformations. Section 2 includes criteria for the selection of design waves. Section 3 gives general planning and structural design principles. Section 4 presents design procedures for rubble-mound structures. Section 5 is a discussion on wave forces on walls and wall design procedures. Section 6 includes applications of floating breakwaters. Section 7 is a discussion of wave forces on cylindrical piles.

For sale by the Superintendent of Documents, U.S. Government Printing Office
Washington, D.C. 20402

FOREWORD

This design manual is one of a series developed from an evaluation of facilities in the shore establishment, from surveys of the availability of new materials and construction methods, and from selection of the best design practices of the Naval Facilities Engineering Command, other Government agencies, and the private sector. This manual uses, to the maximum extent feasible, national professional society, association, and institute standards in accordance with NAVFACENGCOM policy. Deviations from these criteria should not be made without prior approval of NAVFACENGCOM Headquarters (Code 04).

Design cannot remain static any more than can the naval functions it serves or the technologies it uses. Accordingly, recommendations for improvement are encouraged from within the Navy and from the private sector and should be furnished to NAVFACENGCOM Headquarters (Code 04). As the design manuals are revised, they are being restructured. A chapter or a combination of chapters will be issued as a separate design manual for ready reference to specific criteria.

This publication is certified as an official publication of the Naval Facilities Engineering Command and has been reviewed and approved in accordance with SECNAVINST 5600.16.

W. M. Zobel
Rear Admiral CEC, U. S. Navy
Commander
Naval Facilities Engineering Command

HARBOR AND COASTAL FACILITIES DESIGN MANUALS

DM Number	Superseded Chapter in Basic DM-26	Title
26.1	1,4	Harbors
26.2	2	Coastal Protection
26.3	1, 2, 3	Coastal Sedimentation and Dredging
26.4	5	Fixed Moorings
26.5	6	Fleet Moorings
26.6	7	Mooring Design Physical and Empirical Data

CONTENTS

CONTENTS

CONTENTS

CONTENTS

CONTENTS

FIGURES

CONTENTS

CONTENTS

CONTENTS

CONTENTS

CONTENTS

CONTENTS

TABLES

COASTAL PROTECTION

SECTION 1. INTRODUCTION

1. SCOPE. This manual presents basic information required for the design of coastal protective structures.

2. CANCELLATIONS. This manual, NAVFAC DM-26.2, Coastal Protection, cancels and supersedes Chapter 2 of the basic Design Manual 26, Harbor and Coastal Facilities, dated July 1968, and Change 1, dated December 1968.

3. RELATED CRITERIA. Certain criteria related to coastal protection appear elsewhere in the design manual series. See the following sources:

Subject	Source
Cargo Handling Facilities	DM-25.3, DM-38
Channel Layout	DM-26.1
Clear Width of Slips Between Piers, Length of Berth, and Width of Piers	DM-25.1
Coastal Sedimentation and Dredging	DM-26.3
Harbors	DM-26.1
Operational Structures	
Piers and wharves	DM-25.1
Ferry slips, degaussing and deperming facilities, and small-craft berths	DM-25.5
Port Control Offices	DM-23
Quayage Requirements	DM-25.1
Seawalls, Bulkheads, and Quaywalls	DM-25.4
Soil Mechanics, Foundations, and Earth Structures	DM-7
Utilities	DM-3, DM-4, DM-5, DM-25.2

4. GENERAL.

a. Approaches. Waves can be described by deterministic or by spectral theories. In the deterministic approach, the properties of a single wave are used for design. In the spectral approach, the random nature of waves is taken into account. The state-of-the-art of incorporating the spectral approach into engineering is rapidly developing. However, the deterministic approach is presently in widest use in the United States and is the approach which will be followed in the manual. Methods are presented to take into account some of the random properties of wave systems.

b. Wave Classifications. Gravity waves are primarily classified as seas or swell. Seas are waves caused by the wind at the place and time of observation. Swell are waves that have traveled out of the area in which they were generated. Other wave classifications include ship-generated waves, astronomical tides, storm surges, harbor seiches, tsunamis, capillary waves, and internal waves. However, the primary wave considered in the design-of coastal structures is the wind-generated gravity wave having a period ranging from about 1 to 30 seconds.

5. WAVE THEORY. Most coastal engineering design procedures rely on the application of the linear, or "Airy," wave theory, aided by empirically developed procedures for specific design applications. Linear or "Airy" theory provides a first-order approximation to the complete mathematical description of a wave, whereas nonlinear wave theory provides a higher order of approximation. Unfortunately, the higher order of approximation requires a significantly larger mathematical and computational effort. Hence, linear wave theory is often used to the limit of its accuracy. In certain cases, such as wave forces on piles, more sophisticated wave theories are used to account for nonlinear properties of water waves.

6. WAVE PARAMETERS.

a. Definitions. The wave height, H, is the vertical distance between the crest and trough. The wavelength, L, is the distance between two successive wave crests. Wave period, T, is the elapsed time required for two successive wave crests to pass a given point. Wave celerity, or phase velocity, C, is given by L/T. The group velocity, C_g, is the velocity at which the wave group propagates. [eta] is the water-surface elevation at a given point relative to the still water level (SWL), and a is the amplitude, which is equal to H/2. Another useful parameter is the wave steepness, H/L. See Figure 1 for a definition of terms.

b. Relative Depth. Waves can be categorized as shallow-water waves, transitional-water waves, or deepwater waves, depending upon the value of the dimensionless parameter, d/L (relative depth), where d is the still water depth; still water depth is the depth in the absence of waves. Table 1 presents mathematical expressions, categorized by relative depth, for various wave parameters. Throughout the text, the subscript "o" refers to the deepwater value of a wave parameter.

EXAMPLE PROBLEM 1

Given: a. Wave height, H = 10 feet
b. Water depth, d = 20 feet
c. Wavelength, L = 100 feet

Find: Wave steepness, H/L, and relative depth, d/L.

Solution: H/L - 10/100 = 0.100

d/L 20/100 = 0.200

c. Wavelength. The wavelength, L, for a given water depth, d, can be determined graphically by first computing the deepwater wavelength, L , from:

$$L_o = (g/2[pi]) T^2 \qquad (1-1)$$

WHERE: L_o = deepwater wavelength, in feet

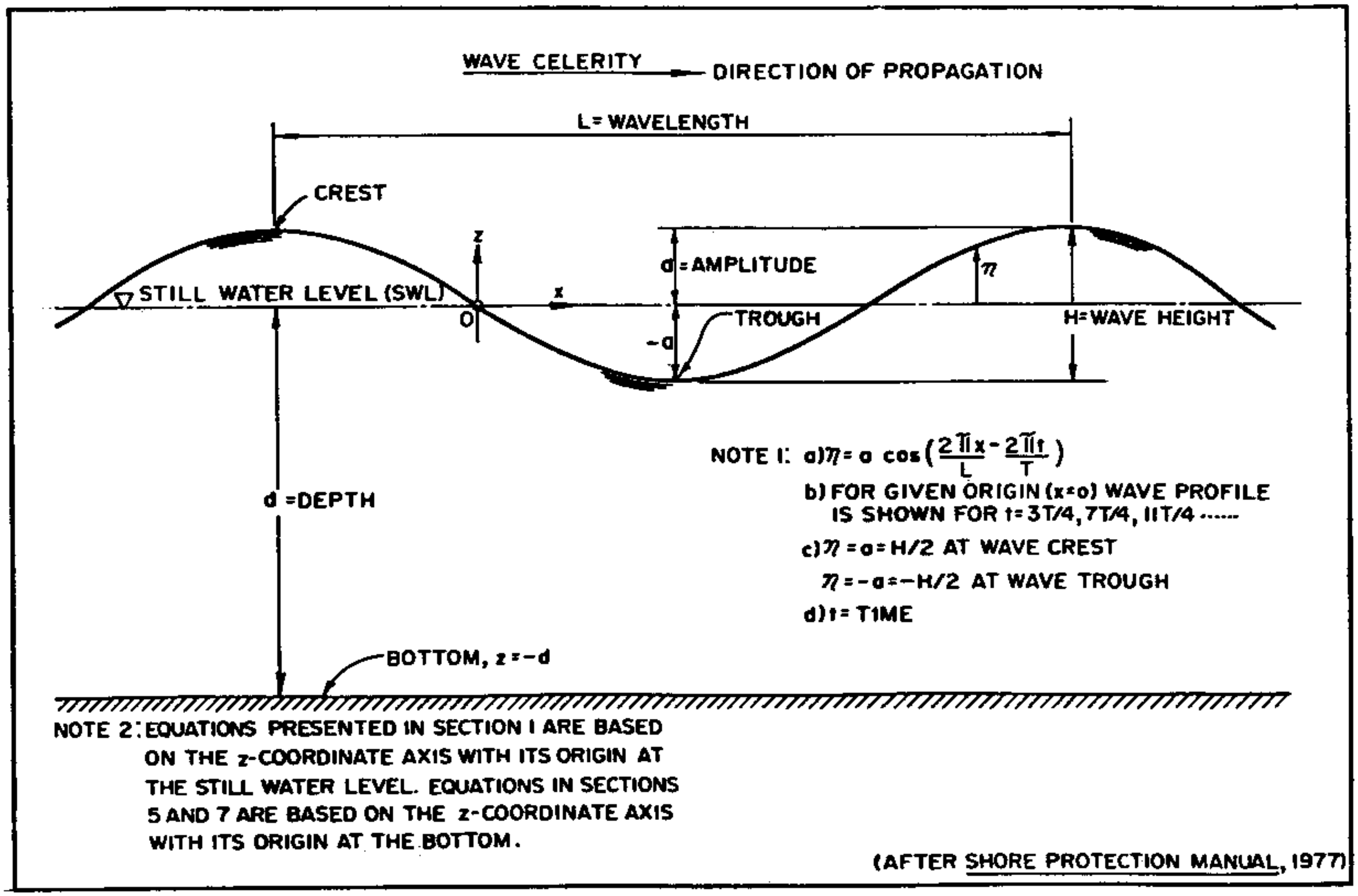


FIGURE 1
Wave Terminology

TABLE 1

Mathematical Expressions, Categorized by Relative Depth, for Various Wave Parameters

Linear (Airy) Wave Theory

RELATIVE DEPTH.....	SHALLOW WATER $\frac{d}{L} < \frac{1}{25}$	TRANSITIONAL WATER $\frac{1}{25} < \frac{d}{L} < \frac{1}{2}$	DEEP WATER $\frac{d}{L} > \frac{1}{2}$
WAVE PARAMETER			
1. Wave Profile	Same As →	$\eta = \frac{H}{2} \cos\left[\frac{2\pi x}{L} - \frac{2\pi t}{T}\right] = \frac{H}{2} \cos\theta$	← Same As
2. Wave Celerity	$C = \frac{L}{T} = \sqrt{gd}$	$C = \frac{L}{T} = \frac{gT}{2\pi} \tanh\left(\frac{2\pi d}{L}\right)$	$C = C_0 = \frac{L}{T} = \frac{gT}{2\pi}$
3. Wave Length	$L = T\sqrt{gd} = CT$	$L = \frac{gT^2}{2\pi} \tanh\left(\frac{2\pi d}{L}\right)$	$L = L_0 = \frac{gT^2}{2\pi} = C_0 T$
4. Group Velocity	$C_g = C = \sqrt{gd}$	$C_g = nC = \frac{1}{2}\left[1 + \frac{4\pi d/L}{\sinh(4\pi d/L)}\right] C$	$C_g = \frac{1}{2} C = \frac{gT}{4\pi}$
5. Water Particle Velocity a) Horizontal	$u = \frac{H}{2}\sqrt{\frac{g}{d}} \cos\theta$	$u = \frac{H}{2} \frac{gT}{L} \frac{\cosh[2\pi(z+d)/L]}{\cosh(2\pi d/L)} \cos\theta$	$u = \frac{\pi H}{T} e^{\frac{2\pi z}{L}} \cos\theta$
b) Vertical	$w = \frac{H\pi}{T}\left(1 + \frac{z}{d}\right) \sin\theta$	$w = \frac{H}{2} \frac{gT}{L} \frac{\sinh[2\pi(z+d)/L]}{\cosh(2\pi d/L)} \sin\theta$	$w = \frac{\pi H}{T} e^{\frac{2\pi z}{L}} \sin\theta$
6. Water Particle Accelerations a) Horizontal	$a_x = \frac{H\pi}{T}\sqrt{\frac{g}{d}} \sin\theta$	$a_x = \frac{g\pi H}{L} \frac{\cosh[2\pi(z+d)/L]}{\cosh(2\pi d/L)} \sin\theta$	$a_x = 2H\left(\frac{\pi}{T}\right)^2 e^{\frac{2\pi z}{L}} \sin\theta$
b) Vertical	$a_z = -2H\left(\frac{\pi}{T}\right)^2\left(1 + \frac{z}{d}\right) \cos\theta$	$a_z = -\frac{g\pi H}{L} \frac{\sinh[2\pi(z+d)/L]}{\cosh(2\pi d/L)} \cos\theta$	$a_z = -2H\left(\frac{\pi}{T}\right)^2 e^{\frac{2\pi z}{L}} \cos\theta$
7. Water Particle Displacement a) Horizontal	$\xi = -\frac{HT}{4\pi}\sqrt{\frac{g}{d}} \sin\theta$	$\xi = -\frac{H}{2} \frac{\cosh[2\pi(z+d)/L]}{\sinh(2\pi d/L)} \sin\theta$	$\xi = -\frac{H}{2} e^{\frac{2\pi z}{L}} \sin\theta$
b) Vertical	$\zeta = \frac{H}{2}\left(1 + \frac{z}{d}\right) \cos\theta$	$\zeta = \frac{H}{2} \frac{\sinh[2\pi(z+d)/L]}{\sinh(2\pi d/L)} \cos\theta$	$\zeta = \frac{H}{2} e^{\frac{2\pi z}{L}} \cos\theta$
8. Subsurface Pressure	$p = \rho g(\eta - z)$	$p = \rho g \eta \frac{\cosh[2\pi(z+d)/L]}{\cosh(2\pi d/L)} - \rho g z$	$p = \rho g \eta e^{\frac{2\pi z}{L}} - \rho g z$

(AFTER SHORE PROTECTION MANUAL, 1977)

for Various Wave Parameters Linear (Airy) Wave Theory] g = gravitational acceleration (32.2 feet per secondÀ2Ù)

T = wave period, in seconds

The value thus determined for LÚo¿is used to determine d/LÚo¿. Figure 2 gives the value of d/L and other parameters as a function of d/LÚo¿. The other parameters will be discussed later. (More accuracy can be obtained by the use of Table C-1 of Appendix C, Shore Protection Manual (1977) (SPM), or by the use of tables found in other wave-theory texts. However, adequate accuracy in most design situations can be obtained by the use of Figure 2.) From Figure 2, the value of d/L for the determined value of d/LÚo¿ may be found; from d/L, the L for the given depth, d, may be calculated. The hyperbolic functions tank(x), sinh(x), and cosh(x), which need to be computed for many of the equations found in Table 1, may be found in Figure 3.

EXAMPLE PROBLEM 2

Given: a. Wave period, T = 10 seconds
b. Water depth, d = 20 feet

Find: Wavelength, L, for d = 20 feet

Solution: (1) Using Equation (1-1), find the deepwater wavelength:

LÚo¿ = (g/2[pi]) TÀ2Ù = (32.2/2[pi])(10)À2Ù = 512 feet

(2) Determine d/LÚo¿:

d/LÚo¿ = 20/512 = 0.039

(3) From Figure 2 for d/LÚo¿ = 0.039:

d/L = 0.082

L = d/0.082 = 20/0.082 = 244 feet

7. WAVE TRANSFORMATIONS. As waves propagate from deep water into intermediate (transitional) and shallow waters, their properties are transformed. The wave period is assumed to remain constant during these transformations. The wave height first decreases relative to the deepwater wave height, HÚo¿, then increases rapidly with a decrease in water depth, d, until breaking occurs. The change in wave height as a function of water depth is termed "wave shoaling." Waves also change height and direction of propagation by wave refraction. Upon encountering a breakwater, waves propagate into the lee of the structure by wave diffraction. Waves propagating in deep water over long distances attenuate in height by dispersion and viscous dissipation. In transitional and shallow water, waves decay due to breaking, bottom friction, and percolation. Waves break when the wave steepness, H/L, approaches about 0.14, or when the wave height relative to water depth, H/d, is from 0.70 to 1.2, depending upon bottom slope. Waves also reflect off beaches, shorelines, and structures.

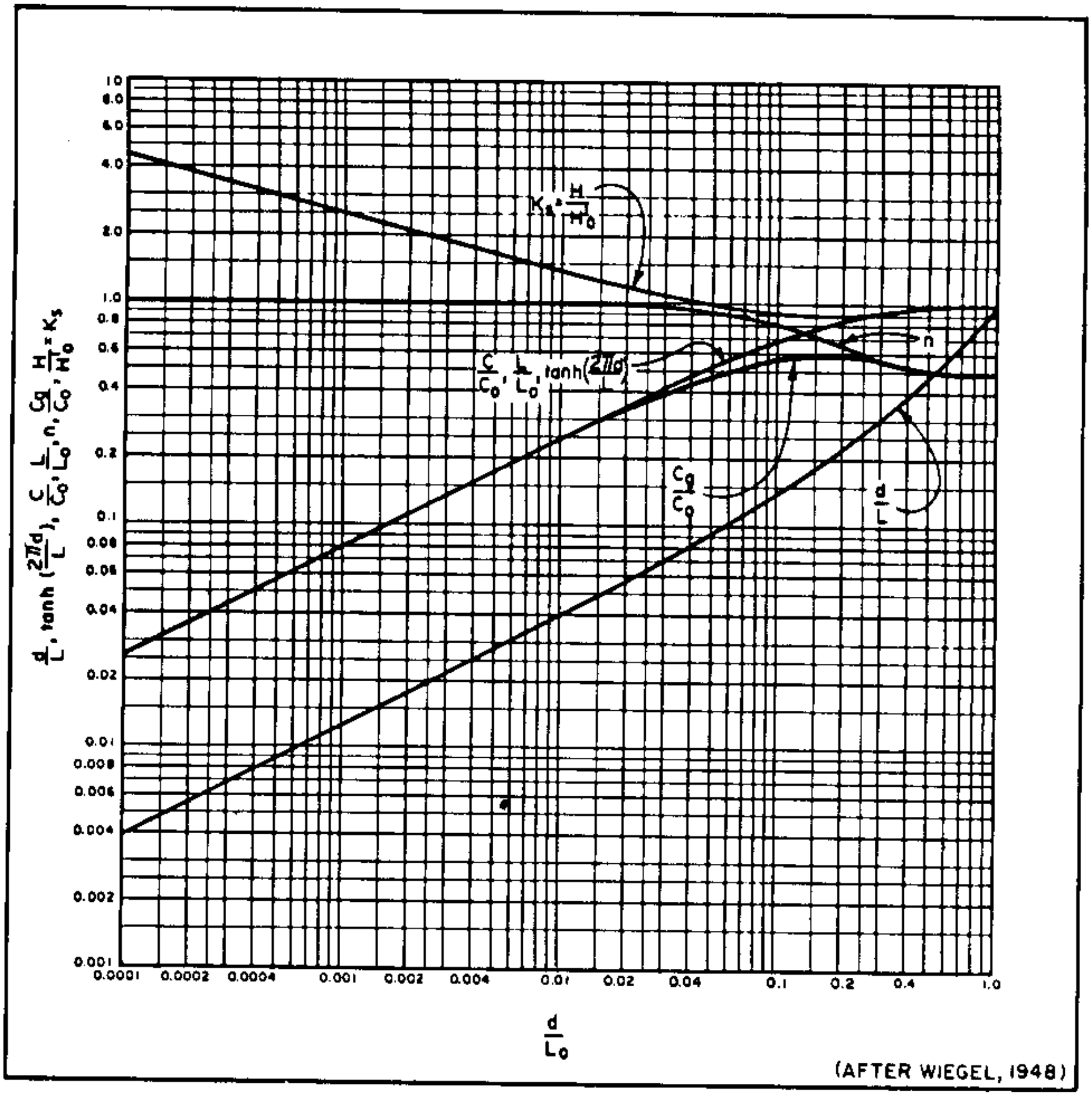


FIGURE 2

Value of Various Wave Parameters as a Function of d/L_o for Linear Wave Theory

Linear Wave Theory]

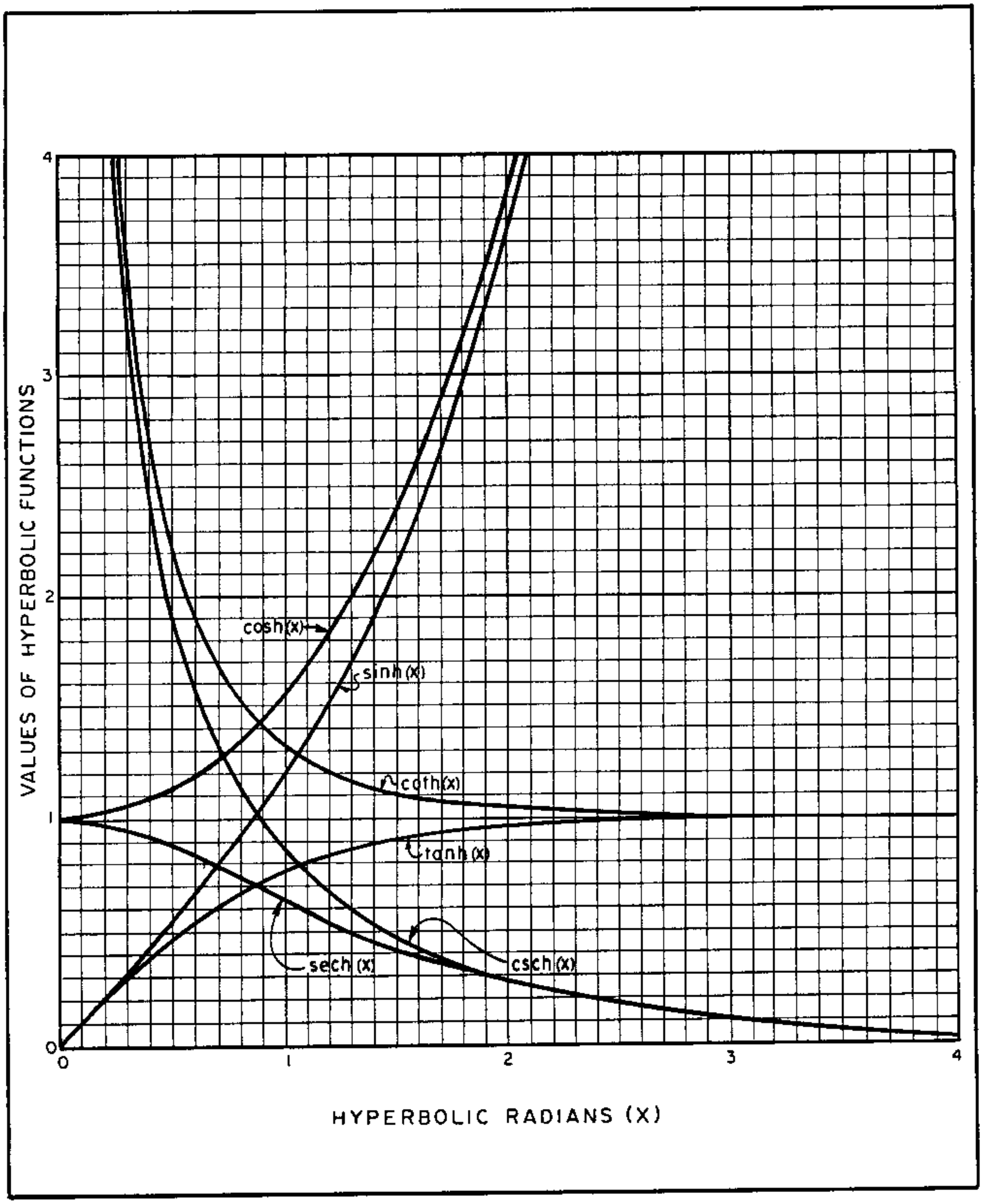


FIGURE 3
Hyperbolic Functions

The wave height, H, at a given location is the product of the shoaling, K_s, refraction, K_R, diffraction, K', and decay, K_f, coefficients, as given by the equation:

$$H = K_s K_R K' K_f H_o \qquad (1\text{-}2)$$

WHERE: H = local wave height

K_s = shoaling coefficient

K_R = refraction coefficient

K' = diffraction coefficient

K_f = decay coefficient

H_o = deepwater wave height

Methods for determining the values for these coefficients and breaking wave heights are presented in the following subsections. The maximum value of H is limited by breaking.

a. Wave Shoaling.

(1) Linear Shoaling. The change in wave height due to a wave entering transitional or shallow water can be determined by application of the shoaling coefficient, $K_s = H/H'_o$, where H'_o represents the equivalent deepwater wave height if the wave had been unaffected by refraction ($H'_o = H_o K_R$). Figure 2 shows a plot of the first-order approximation of the linear shoaling coefficient, H/H'_o, as a function of d/L_o.

(2) Nonlinear Shoaling. As the wave approaches very shallow water several wavelengths seaward of breaking, shoaling becomes highly nonlinear, and the linear shoaling coefficient may significantly underpredict the wave height, especially for long waves in shallow water. Figure 4 gives an approximation of the nonlinear shoaling coefficient, K_{sNL}, for values of deepwater wave steepness, H'_o/L_o, versus relative depth, d/L_o. The lines of slope, m, are used to determine whether breaking has occurred for the given set of conditions. Also plotted on Figure 4 is the linear shoaling coefficient, K_s, which is given by the curve denoted m = 0. This graph should be used as a check to determine the relative importance of nonlinear properties on the shoaling of a given wave. Figure 4 plots the nonlinear shoaling coefficient, K_{sNL}, as a function of d/L_o for isolines of H'_o/L_o. To find an appropriate shoaling coefficient, enter the abscissa of Figure 4 with a given value of d/L_o. Proceed, extending a vertical line from the d/L_o value, until intersection with the given value-of H'_o/L_o. If the lines do not intersect and the H'_o/L_o value lies to the let of the d/L_o value, then the nonlinear properties of the wave are not affecting wave shoaling in the given water depth; in that case, the linear shoaling coefficient, K_s, denoted by the m = 0 line, would be used. If the lines do not intersect and the H'_o/L_o value lies to the right of the given d/L_o value, then the wave has already broken in deeper water. Where the line extending from the given d/L_o and the given H'_o/L_o do intersect, a horizontal line is extended to the ordinate to obtain type value of the nonlinear shoaling coefficient, K_{sNL}.

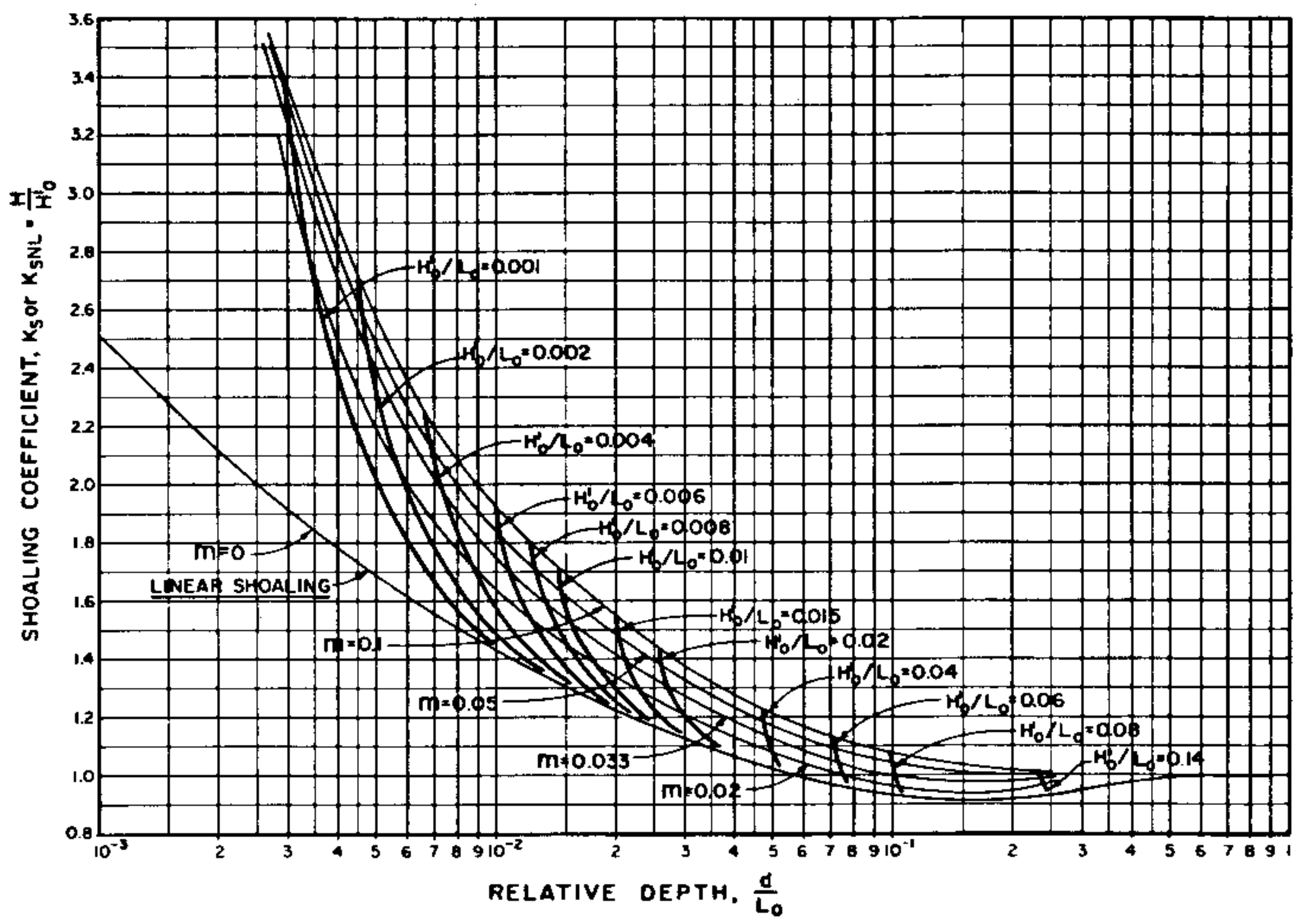


FIGURE 4
Shoaling Coefficient, K_s or K_{sNL}

However, if the given value of m lies below the intersection of the given d/L_o and H'_o/L_o , then the wave has broken and the wave will not shoal as high as the K_{sNL} value indicates. If the given value of m lies above the intersection of given d/L_o and H'_o/L_o, then the indicated K_{sNL} should be used. The lines labeled with various values of slope (in addition to m = 0) indicate the breaking limits (indices) for a given m, d/L_o, and H'_o/L_o. The wave cannot shoal past the breaking limit for a given bottom slope, m. The wave shoals to higher breaking limits for steeper slopes. The region of validity of Figure 4 is restricted to the region just prior to breaking. Figure 4 is a semiempirical plot based on breaking indices described in Subsection 1.6.e, Wave Breaking, and on theoretical nonlinear shoaling curves. The K_{sNL} is only an approximation to account for the discrepancy between linear shoaling and empirical breaking indices. Application of Figure 4 is illustrated in Example Problem 3.

EXAMPLE PROBLEM 3

Given: a. Case I: H_o = 10 feet, T = 12 seconds, d = 11.8 feet, m = 0.033, K_R = 0.74

b. Case II: H_o = 5.75 feet, T = 15 seconds, d = 5.8 feet, m = 0.033, K_R = 0.4

c. Case III: H = 2.5 feet, T = 10 seconds, d = 5.12 feet, m = 0.033

Find: a. Case I: shallow-water wave height, H, using K_{sNL} and compare to H obtained using linear K_s.

b. Case II: shallow-water wave height, H, using K_{sNL} and compare to H obtained using linear K_s.

c. Case III: equivalent unrefracted deepwater wave height, H'_o, using K_{sNL}.

Solution: a. Case I:

(1) Using Equation (1-1), find L_o:

$L = (g/2\pi) T^2 = (32.2/2\pi)(12)^2 = 738$ feet

(2) Determine d/L_o:

$d/L_o = 11.8/738 = 0.016$

(3) Determine H'_o:

$H'_o = H_o K_R = (10)(0.74) = 7.4$ feet

(4) Determine deepwater steepness, H'_o/L_o:

$H'_o/L_o = 7.4/738 = 0.01$

EXAMPLE PROBLEM 3 (Continued)

(5) From Figure 4 for d/LÚo¿ = 0.016, Ho/Lo = 0.01, and m = 0.033:

KÚsNL¿ = 1.48

THEREFORE: Nonlinear value of H = KÚsNL¿ H'Úo¿ = (1.48)(7.4) = 11.0 feet

(6) From Figure 4 for d/LÚo¿ = 0.016, H'Úo¿/LÚo¿ = 0.01, and m = 0 (linear shoaling):

KÚs¿ = 1.28

THEREFORE: From linear theory, H = KÚs¿ H'Úo¿= (1.28)(7.4) = 9.5 feet

THEREFORE: Nonlinear shoaling predicts a wave height that is 16 percent greater than that predicted by linear shoaling.

Note: If the slope had been m = 0.02 instead of 0.033, the wave would have broken at a value of KÚsNL¿ = 1.39.

b. Case II: (1) Using Equation (1-1), find LÚo¿:

LÚo¿ = (g/2[pi]) TÀ2Ù = (32.2/2[pi])(15)À2Ù = 1,153 feet

(2) Determine d/LÚo¿:

d/LÚo¿ = 5.75/1,153 = 0.005

(3) Determine H'Úo¿:

H'Úo¿ = HÚo¿ KÚR¿ = (5.75)(0.4) = 2.3 feet

(4) Determine deepwater steepness, H'Úo¿/LÚo¿:

H'Úo¿/LÚo¿ = 2.3/1,153 = 0.002

(5) From Figure 4 for d/LÚo¿ = 0.005, H'Úo¿/LÚo¿ = 0.002, and m = 0.033:

KÚsNL¿ = 2.33

THEREFORE: Nonlinear value of H = KÚsNL¿ H'Úo¿ = (2.33)(2.3) = 5.36 feet

(6) From Figure 4 for d/LÚo¿ = 0.05, H'Úo¿/LÚo¿ = 0.002, and m = 0:

KÚs¿ = 1.69

EXAMPLE PROBLEM 3 (Continued)

THEREFORE: From linear theory, $H = K_s H'_o = (1.69)(2.3) = 3.89$ feet

THEREFORE: Nonlinear shoaling predicts a wave height that is 38 percent greater than that predicted by linear shoaling.

c. Case III: to determine the value of H'_o requires an iterative process. First assume a value of H'_o/L_o; at the intersection of d/L_o line and assumed H'_o/L_o read value of K_{sNL}; compute H'_o from assumed H'_o/L_o and L_o; use K_{sNL} and H'_o to get a value of H; compare computed H to actual value of H. Repeat the process until the computed H converges with the actual H. Calculate H'_o from the assumed H'_o/L_o which yielded the actual H.

(1) Using Equation (1-1), find L_o:

$L_o = (g/2\pi) T^2 = (32.2/2\pi)(10)^2 = 512$ feet

(2) Determine d/L_o:

$d/L_o = 5.12/512 = 0.010$

(3) First, try $H'_o/L_o = 0.004$:

From Figure 4 for $d/L_o = 0.01$, $H'_o/L_o = 0.004$, and $m = 0.033$:

$K_{sNL} = 1.57$

$H'_o/L_o = 0.004$; $H'_o = 0.004 L_o = (0.004)(512) = 2.05$ feet

$H = K_{sNL} H'_o = (1.57)(2.05)$

$H = 3.22$ feet; since $H = 2.5$ feet, $H'_o/L_o = 0.004$ is too high

(4) Secondly, try $H'_o/L_o = 0.003$:

From Figure 4 for $d/L_o = 0.01$, $H'_o/L_o = 0.003$, and $m = 0.033$:

$K_{sNL} = 1.53$

$H'_o/L_o = 0.003$; $H'_o = (0.003)(512) = 1.54$ feet

$H = K_{sNL} H'_o = (1.53)(1.54)$

$H = 2.36$ feet; since $H = 2.5$ feet, $H'_o/L_o = 0.003$ is too low
(5) Thirdly, try $H'_o/L_o = 0.0032$:

EXAMPLE PROBLEM 3 (Continued)

From Figure 4 for $d/L_o = 0.01, H'_o/L_o = 0.0032$ and $m = 0.033$:

$K_{sNL} = 1.54$

$H'_o/L_o = 0.003$; $H'_o = (0.0032)(512) = 1.64$ feet

$H = K_{sNL} H'_o = (1.54)(1.64)$

$H = 2.53$ feet ÷ 2.5 feet

THEREFORE: $H'_o = 1.64$ feet

b. Wave Refraction.

(1) General. Waves are considered to be in deep water for $d/L_o >$ 1/2; however, when waves propagate into shallower water, the phase velocity, C, becomes a function of water depth. When the wave crests are at an angle relative to the bottom depth contours, the wave crests bend, tending to aline with the depth contours. Figure 5 schematically shows wave refraction over straight and parallel bottom contours. Waves converge over submarine ridges and diverge over submarine canyons, as shown in Figure 6 . Wave orthogonals are imaginary lines drawn perpendicularly to the wave crests which indicate the direction of wave propagation. When the orthogonals converge, the wave height increases proportionally with the refraction coefficient, K_R, which is a function of q square root of the ratio of orthogonal spacing, $K_R = H_2/H_1 = (b_1/b_2)^{1/2}$. (See Figure 5.) (b = distance between orthogonals.) Conversely, when the orthogonals diverge, the wave height decreases.

(2) Importance. Wave refraction and wave shoaling are important wave transformations that affect structural designs and analyses of beach systems. Refraction must be considered in design of structures to determine the angle of wave approach and the change in wave height for waves in transitional and shallow water. For example, the wave-height distribution along a shoreline can be greatly influenced by the offshore bathymetry. A harbor entrance should be located in an area of wave divergence rather than convergence. This will result in a more protected harbor. Wave refraction should be considered in determining such things as breakwater armor-unit sizes, wave-induced forces on piles and other structures, and wave runup. Wave refraction is also an important phenomenon in studying littoral transport and shoreline configurations. (See DM-26.3.)

(3) Refraction Over Straight and Parallel Contours. Refraction effects over a bottom having straight and parallel depth contours can be calculated by application of Figure 7. The refraction coefficient, K_R, and the angle of wave crest relative to the depth contour, a, at a given depth, d, for a given period, T, can e determined by entering Figure 7 on the abscissa with a value of d/gT^2 and the ordinate with a deepwater angle of approach, α_o.

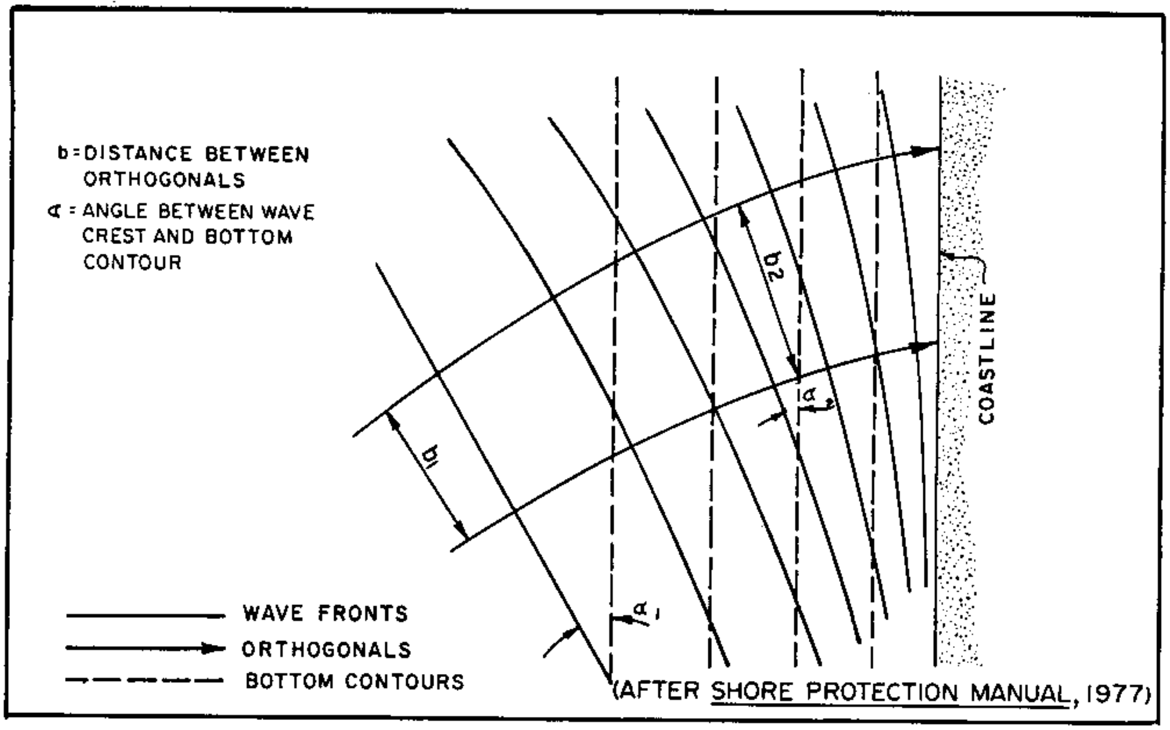


FIGURE 5
Schematic of Wave Refraction Over Straight and Parallel Bottom Contours

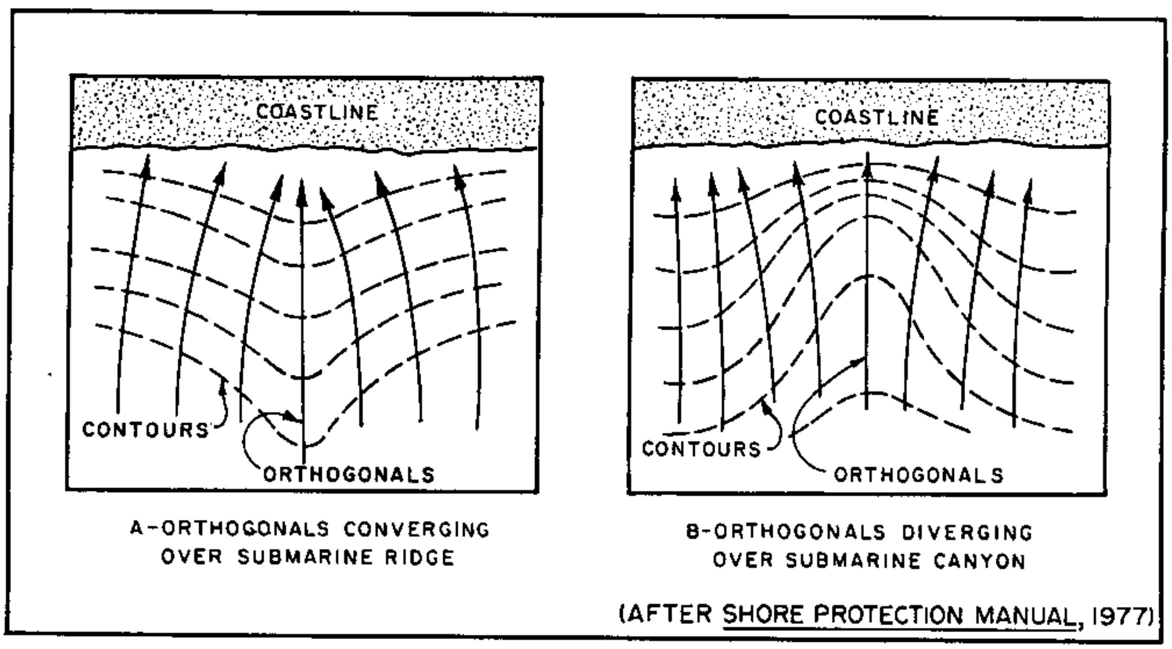
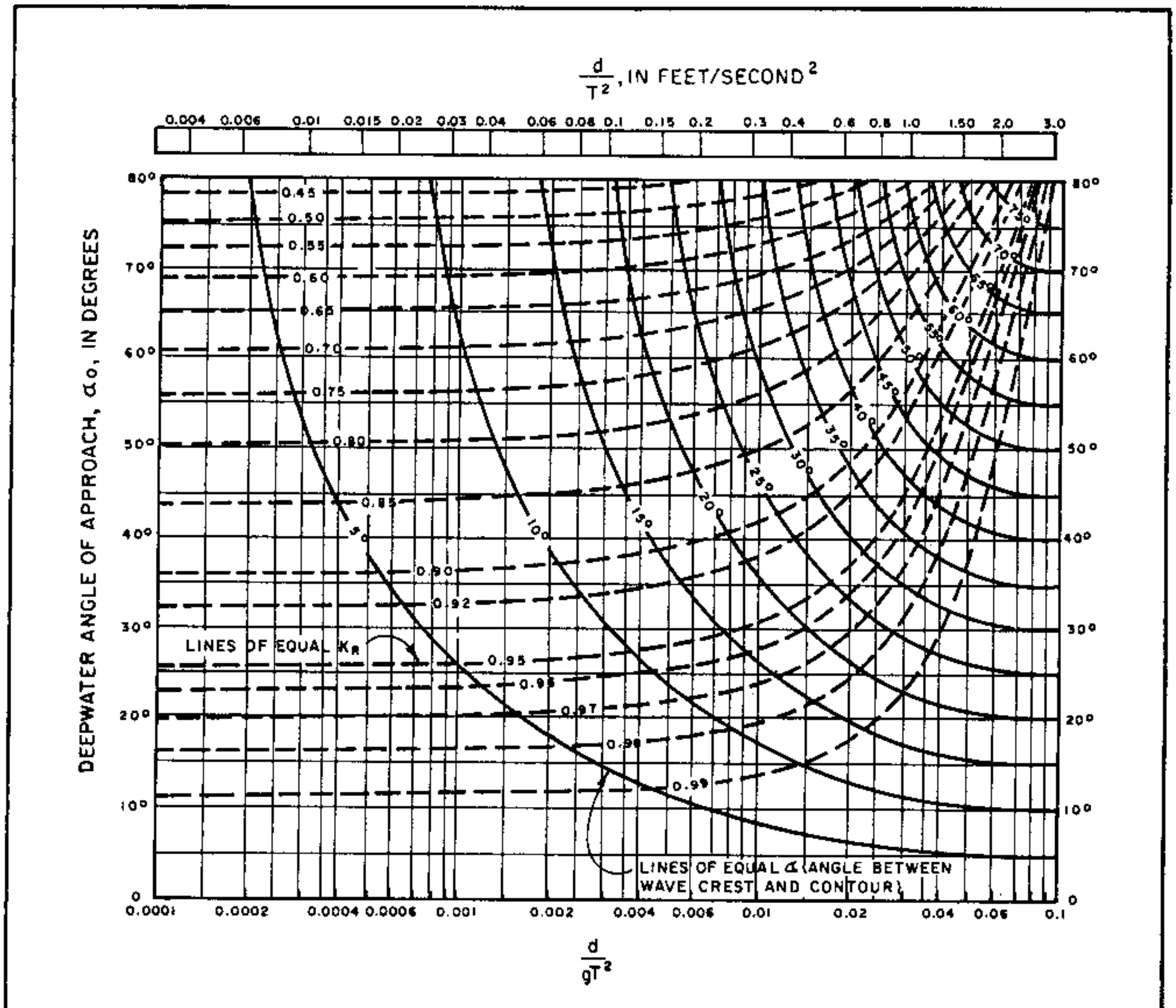


FIGURE 6
Converging and Diverging Wave Refraction

Parallel Bottom Contours & Converging and Diverging Wave Refraction]



(SHORE PROTECTION MANUAL, 1977)

FIGURE 7
Wave Refraction Parameters for Straight and Parallel Contours

Contours]

The local wave height is given by:

$$H = K_s K_R H_o \qquad (1\text{-}3)$$

EXAMPLE PROBLEM 4

Given: a. A beach with straight and parallel contours
b. Incident deepwater wave characteristics:
H = 6 feet and T = 10 seconds with wave crests at a 30 deg. angle relative to the bottom contours

Find: Wave height and direction of wave propagation at d = 30 feet.

Solution: (1) Find d/gT^2:

$d/gT^2 = 30/[(32.2)(10)^2] = 0.0093$

(2) From Figure 7 for α_o = 30 deg. and $d/gT^2 = 0.0093$:

α = 17.0 deg. and $K_R = 0.95$

(3) Find d/L_o:

$L_o = (g/2\pi) T^2 = (32.2/2\pi)(10)^2 = 512$ feet

$d/L_o = 30/512 = 0.0586$

(4) From Figure 2 for $d/L_o = 0.0586$:

$K_s = 1.0$

(5) Using Equation (1-3):

$H = K_s K_R H_o = (1.0)(0.95)(6) = 5.7$ feet

(4) Refraction Over Irregular Bathymetry. Refraction over irregular bathymetry, such as over submarine ridges and canyons, requires the use of graphical methods or computer programs. These methods are described in the Shore Protection Manual (1977).

EXAMPLE PROBLEM 5

Given: Deepwater waves, H_o = 10 feet, T = 10 seconds from the northeast entering Main Pass of Diego Garcia. The shoaling coefficient, K_s = 1.52.

Find: Determine the wave height and angle of incidence at the entrance of a proposed boat harbor in 10 feet of water.

EXAMPLE PROBLEM 5 (Continued)

Solution: The project site is shown in Figure 8 with its bathymetry and refraction diagram. The bathymetry is very complex and the straight-and-parallel-depth-contour assumption is not appropriate. The refraction must be solved by graphical procedures described in the Shore Protection Manual (1977), or by a computer program.

The solution given in Figure 8 shows a severe refraction of incident wave energy around the sloping banks of Main Pass. At the project site, the refraction coefficient is $K_R = 0.22$ as determined by using the distance between orthogonals and the equation $K_R = (b_1/b_2)^{1/2}$, where b_1 is measured in deep water outside the entrance and b_2 is measured near the deep-draft wharf. The shoaling coefficient is $K_s = 1.52$. The wave approaches from the north. The resultant wave height is:

$$H = K_s K_R H_o = (1.52)(0.22)(10) = 3.34 \text{ feet}$$

c. Wave Diffraction. Diffraction of water waves occurs when a wave train is interrupted by a barrier such as the breakwater shown in Figure 9. Waves propagate into the lee of the breakwater essentially in circular arcs radiating from the head of the breakwater. Wave heights in the lee, inside the geometric shadow, are less than one-half the incident wave height. A diffraction diagram describes wave-height distribution in the lee of a breakwater. The diffraction diagram shows isolines of diffraction coefficients, K', for a given local wavelength and angle of approach. The wave height in the vicinity of a breakwater is determined by:

$$H = k' H_i \qquad (1\text{-}4)$$

WHERE: H = local wave height (diffracted wave height)

K' = diffraction coefficient

H_i = incident wave height

Note: The effects of refraction and shoaling must also be included.

Waves also reflect off the obstruction, causing an interference pattern on the seaward side.

(1) Single Semi-Infinite, Rigid, Impermeable breakwater. For a single breakwater, the diffraction coefficient, K', is a function of the angle of wave approach relative to the breakwater, [phi] , and wavelength, L, in water depth, d_s , at the toe of the breakwater head. Figures 10 through 21 give diffraction diagrams for a thin breakwater of semi-infinite length in constant water depth for different angles of wave approach.

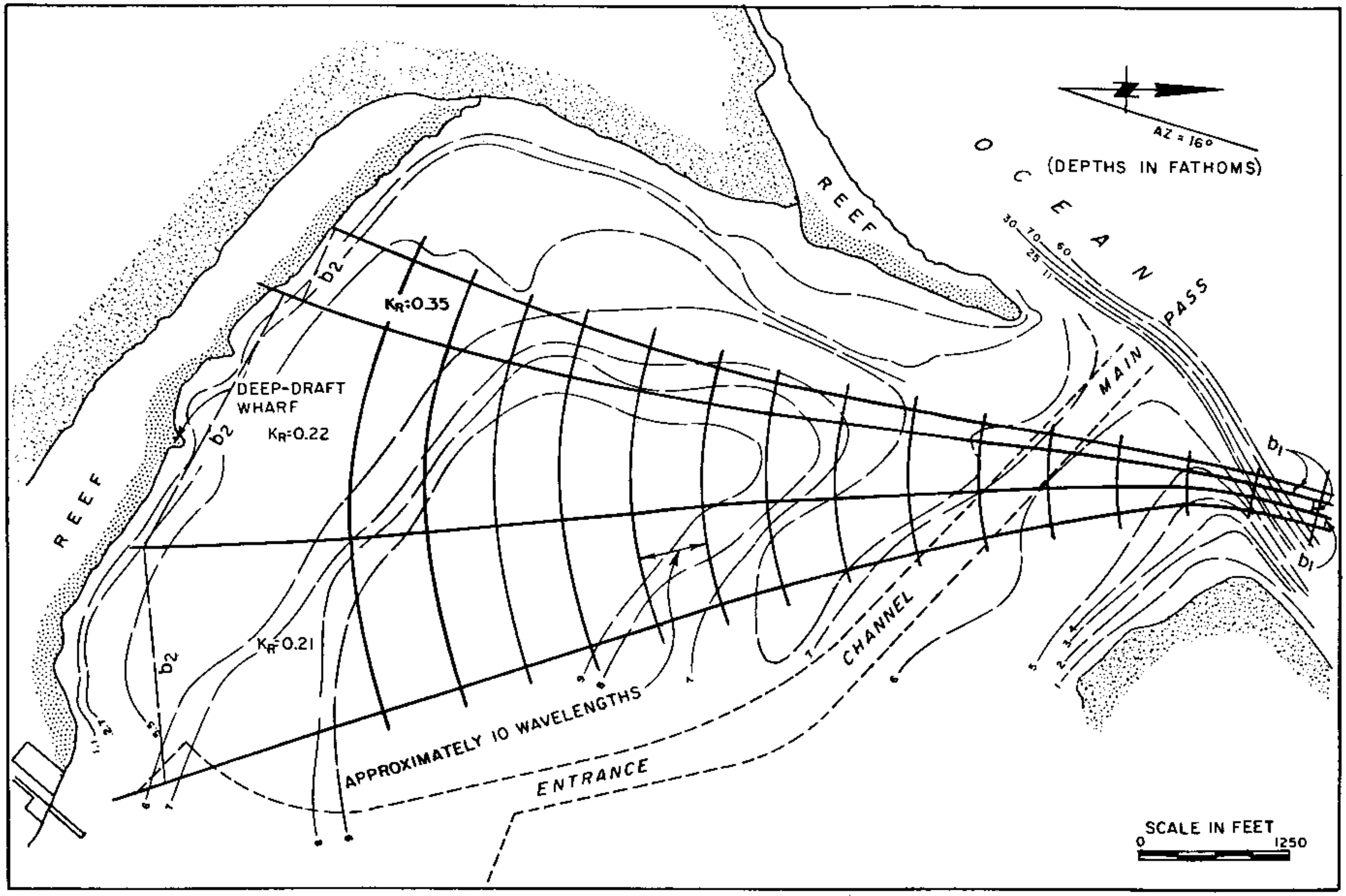


FIGURE 8
Bathymetry, Project Site, and Refraction Diagram for Example Problem 5

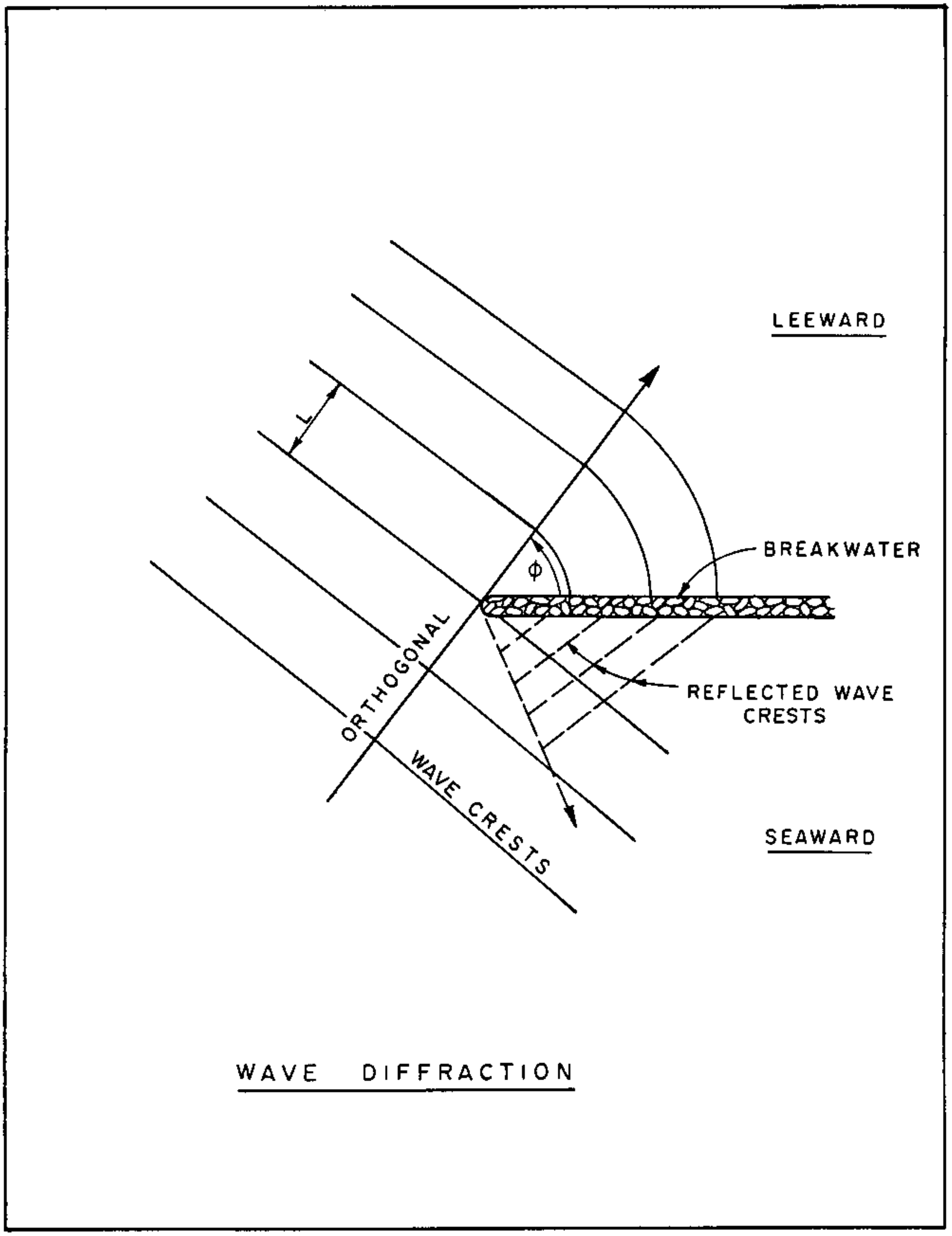


FIGURE 9
Diffraction Occurs When Wave Train is Interrupted by a Breakwater

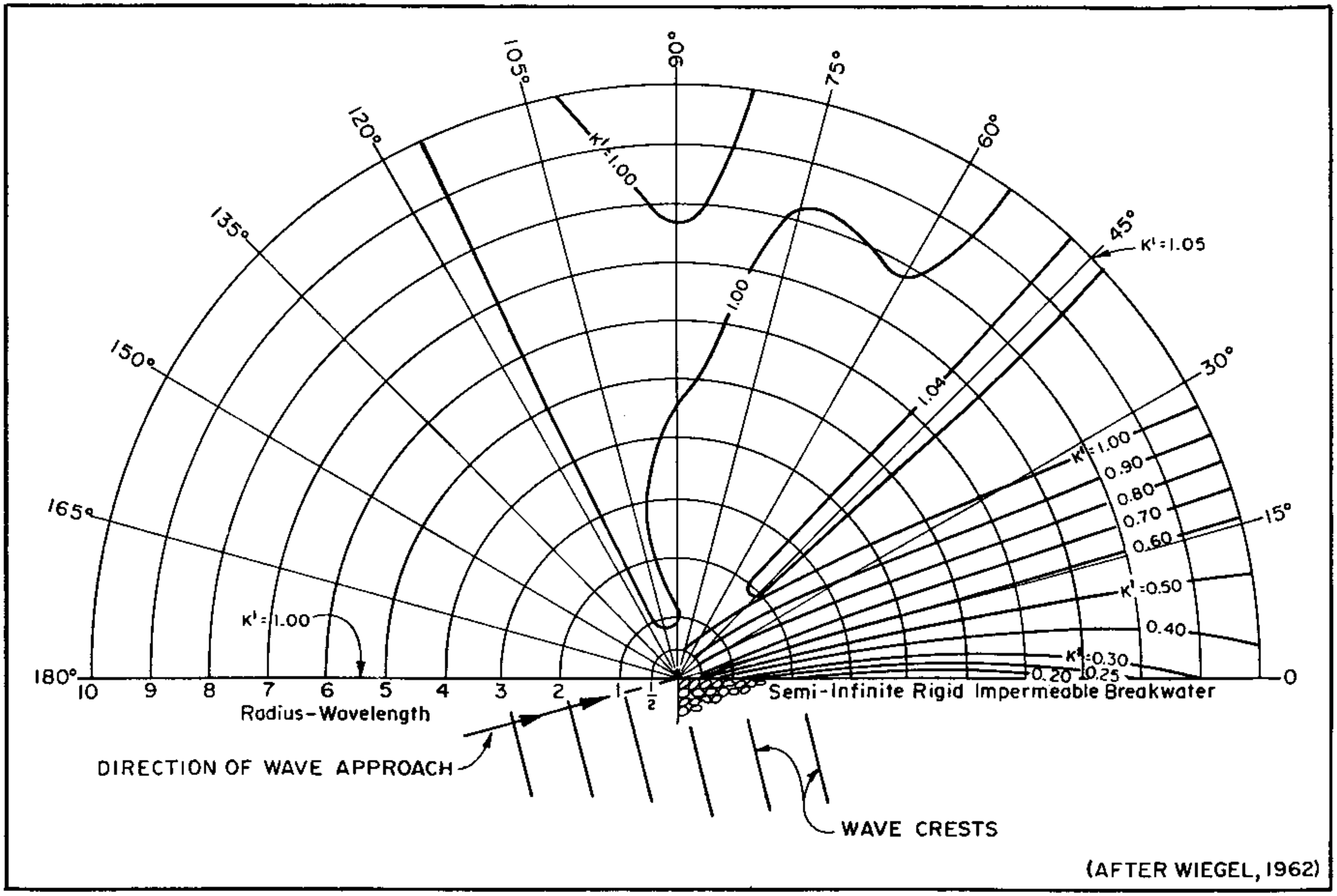


FIGURE 10
Wave-Diffraction Diagram for 15° Angle of Wave Approach

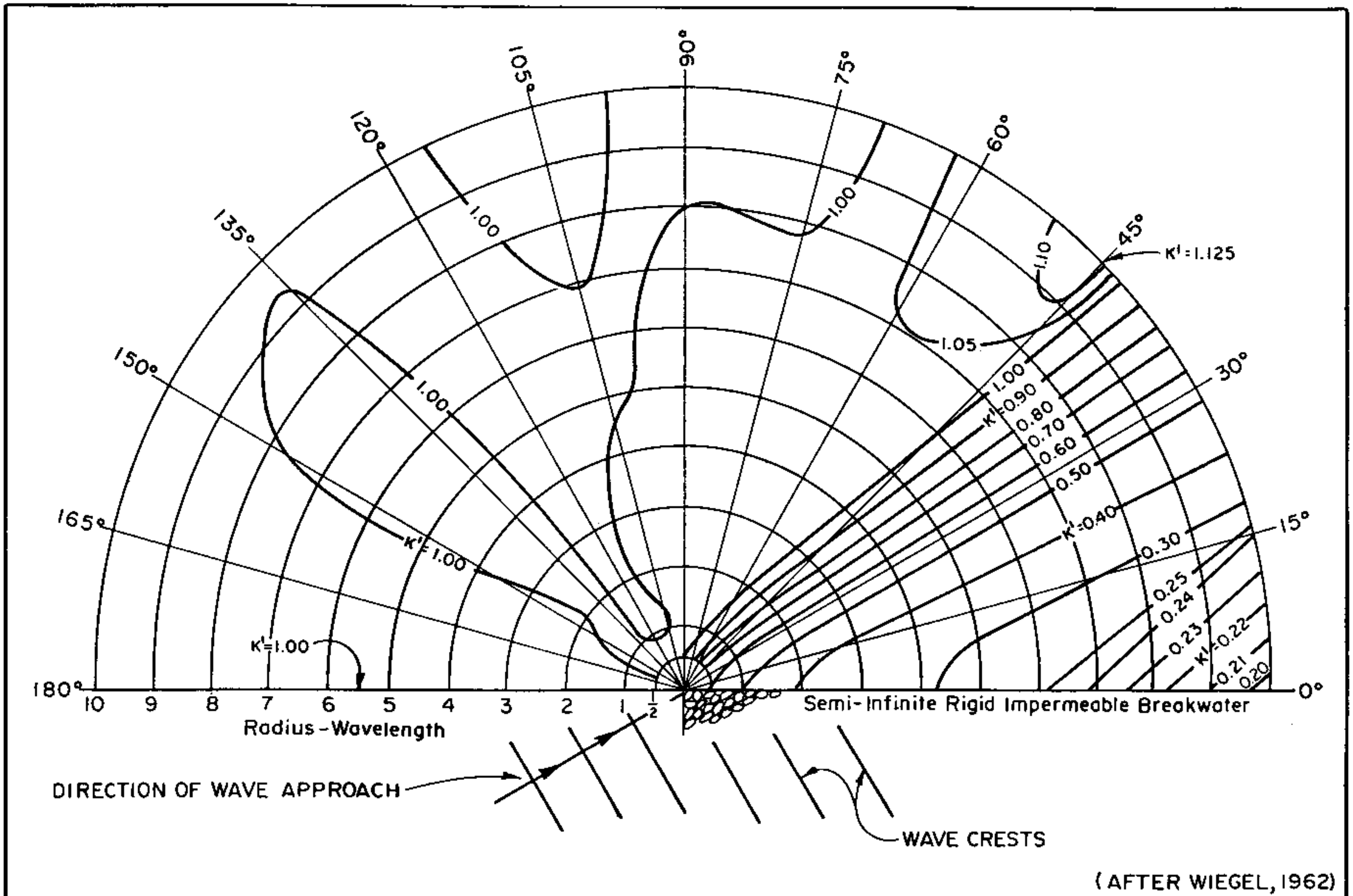


FIGURE 11
Wave-Diffraction Diagram for 30° Angle of Wave Approach

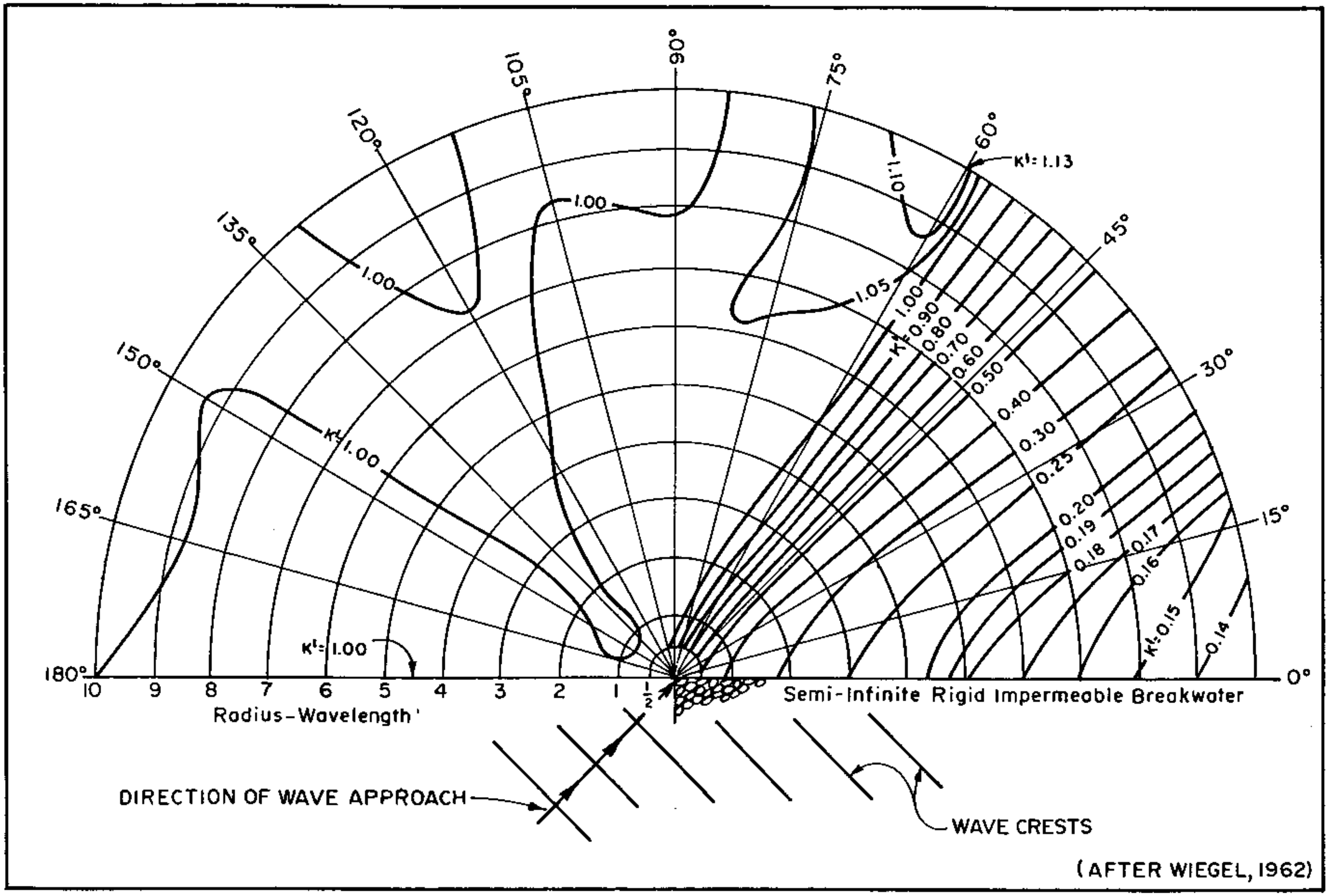


FIGURE 12
Wave-Diffraction Diagram for 45° Angle of Wave Approach

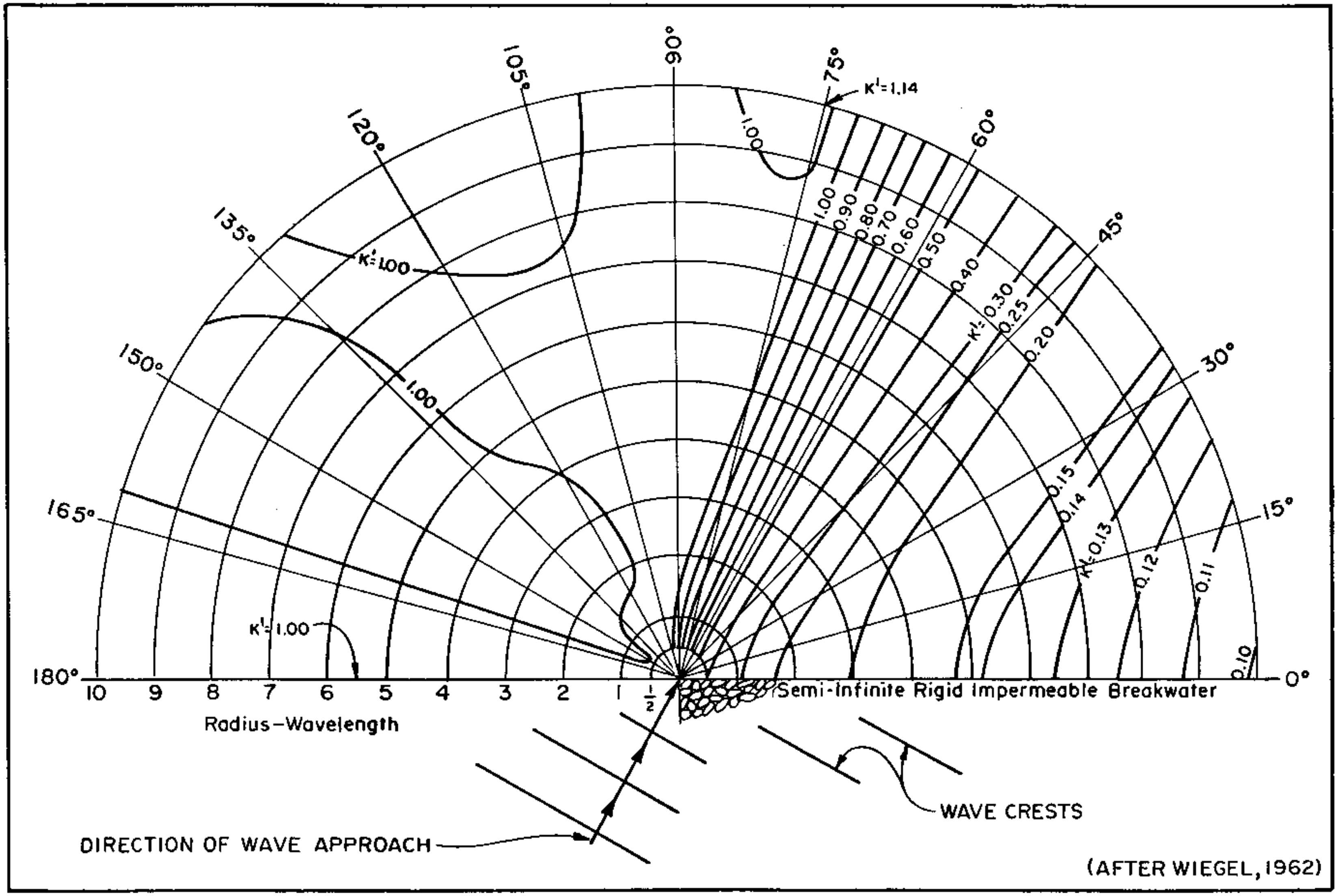


FIGURE 13
Wave-Diffraction Diagram for 60° Angle of Wave Approach

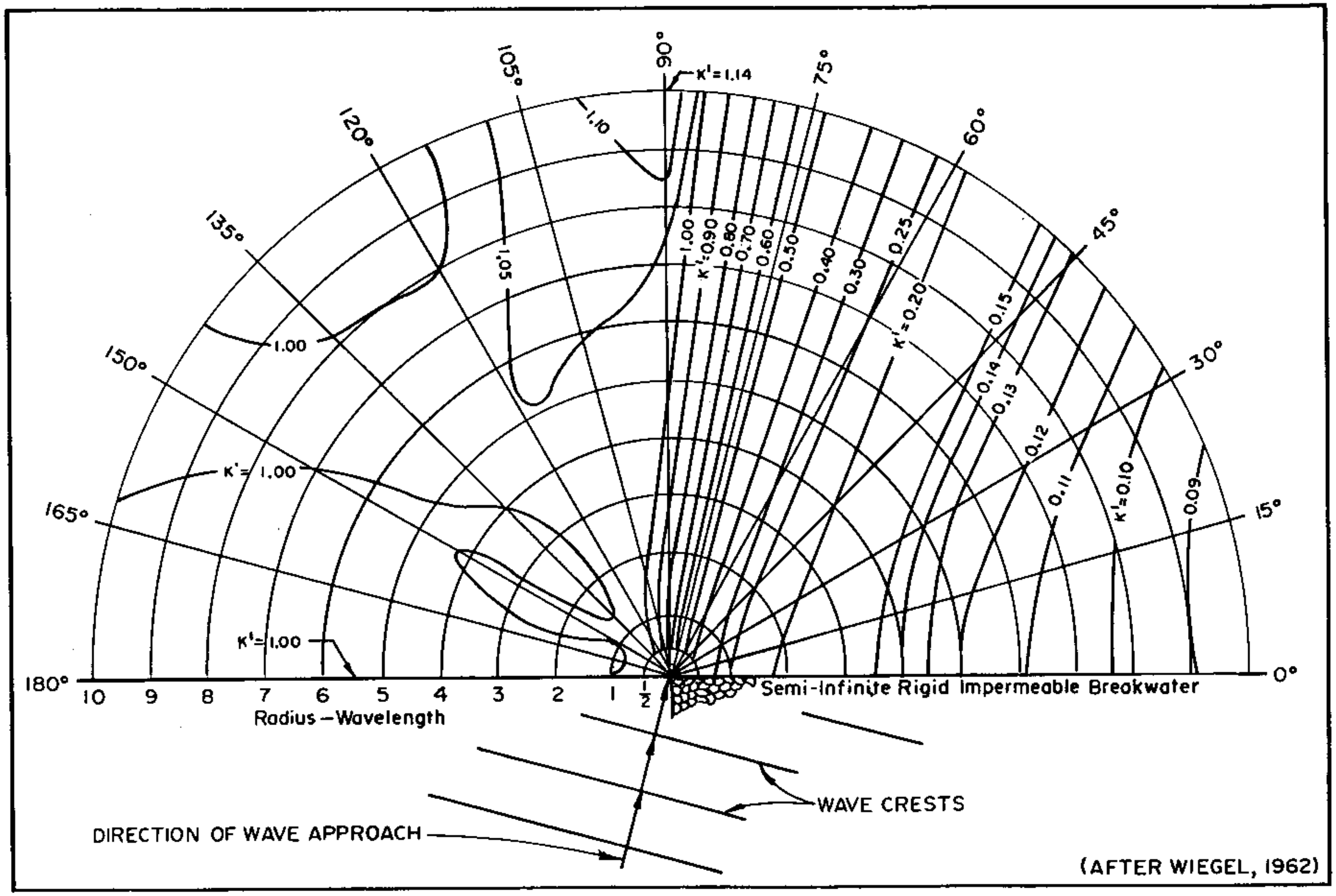


FIGURE 14
Wave-Diffraction Diagram for 75° Angle of Wave Approach

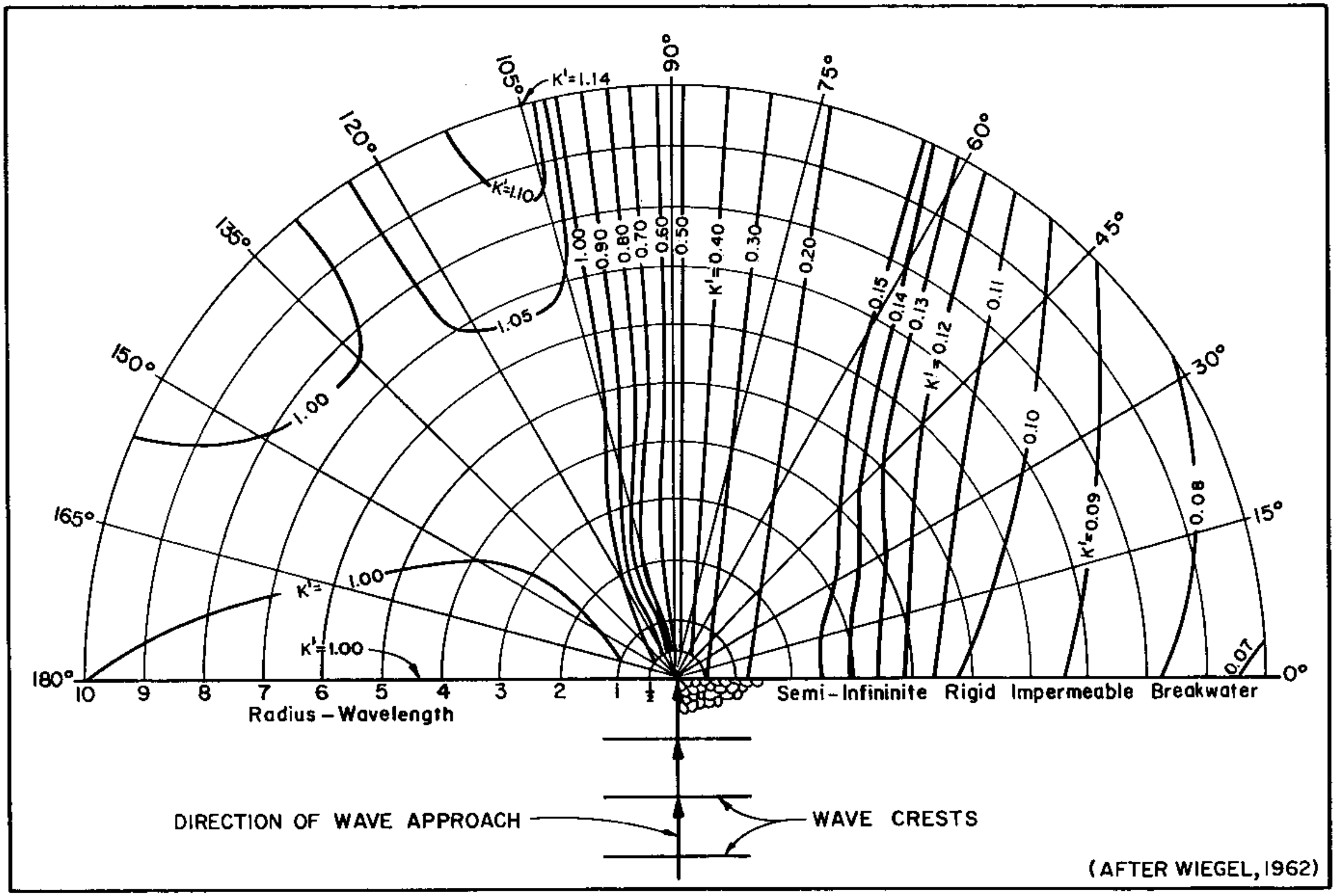


FIGURE 15
Wave-Diffraction Diagram for 90° Angle of Wave Approach

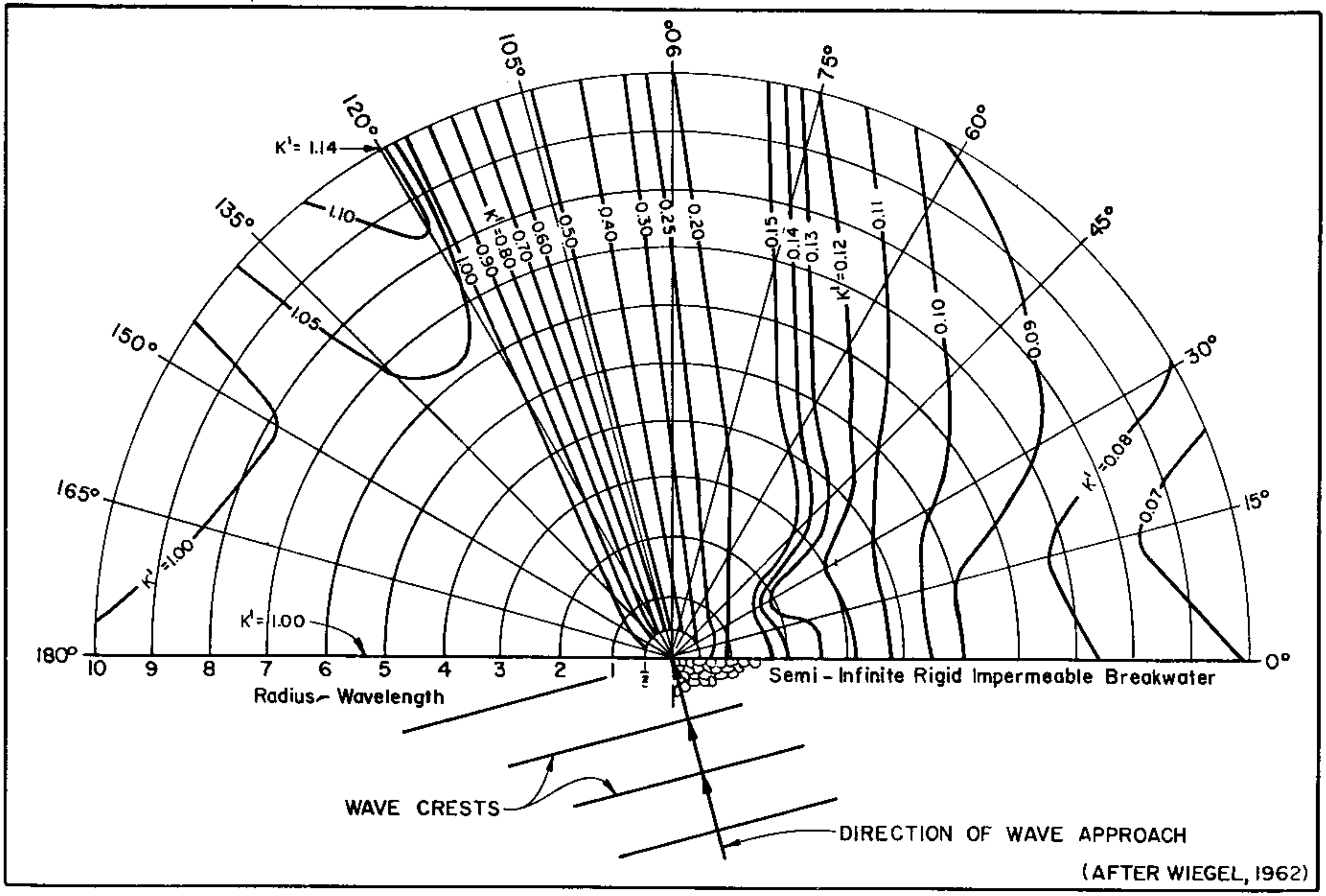


FIGURE 16
Wave-Diffraction Diagram for 105° Angle of Wave Approach

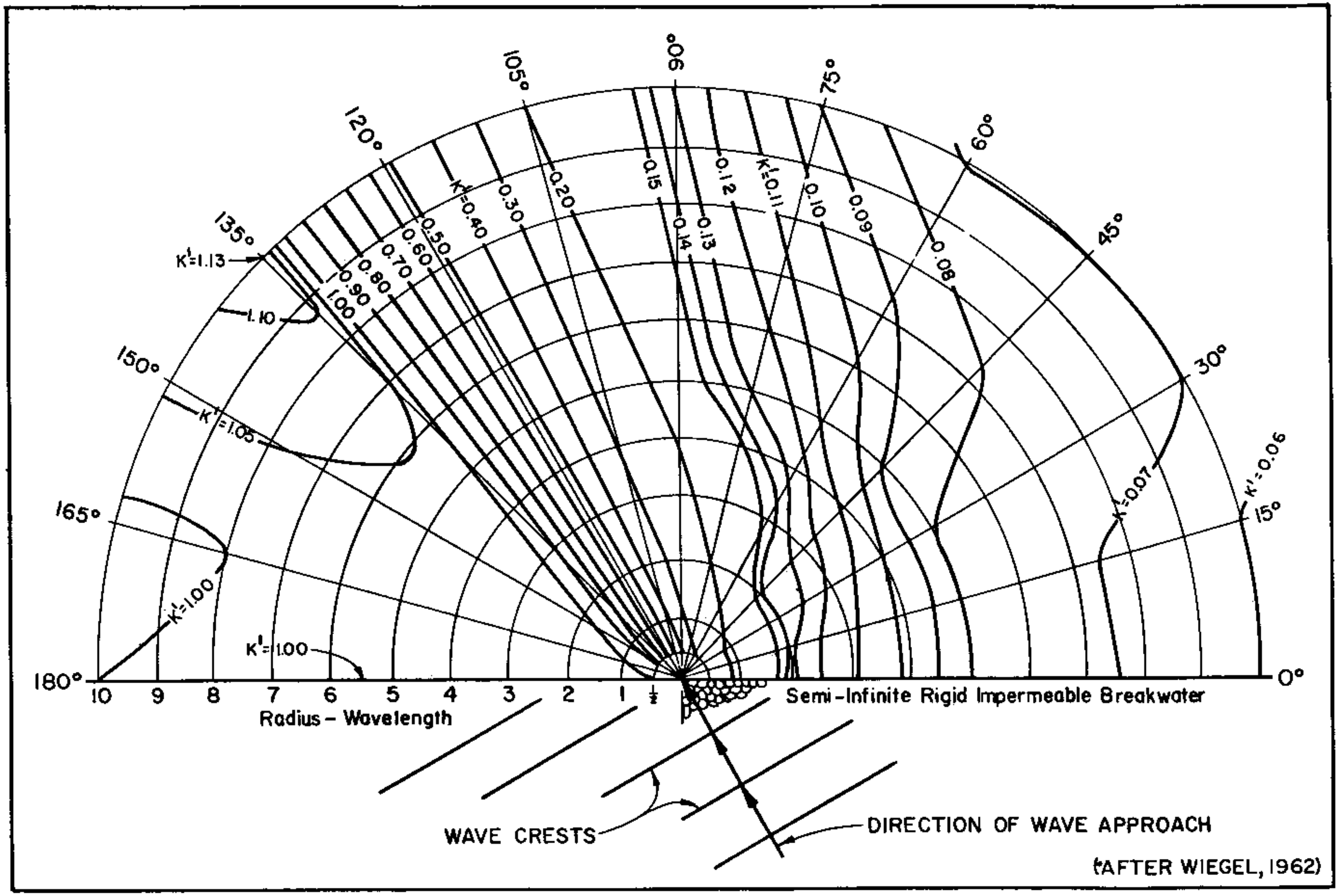


FIGURE 17
Wave-Diffraction Diagram for 120° Angle of Wave Approach

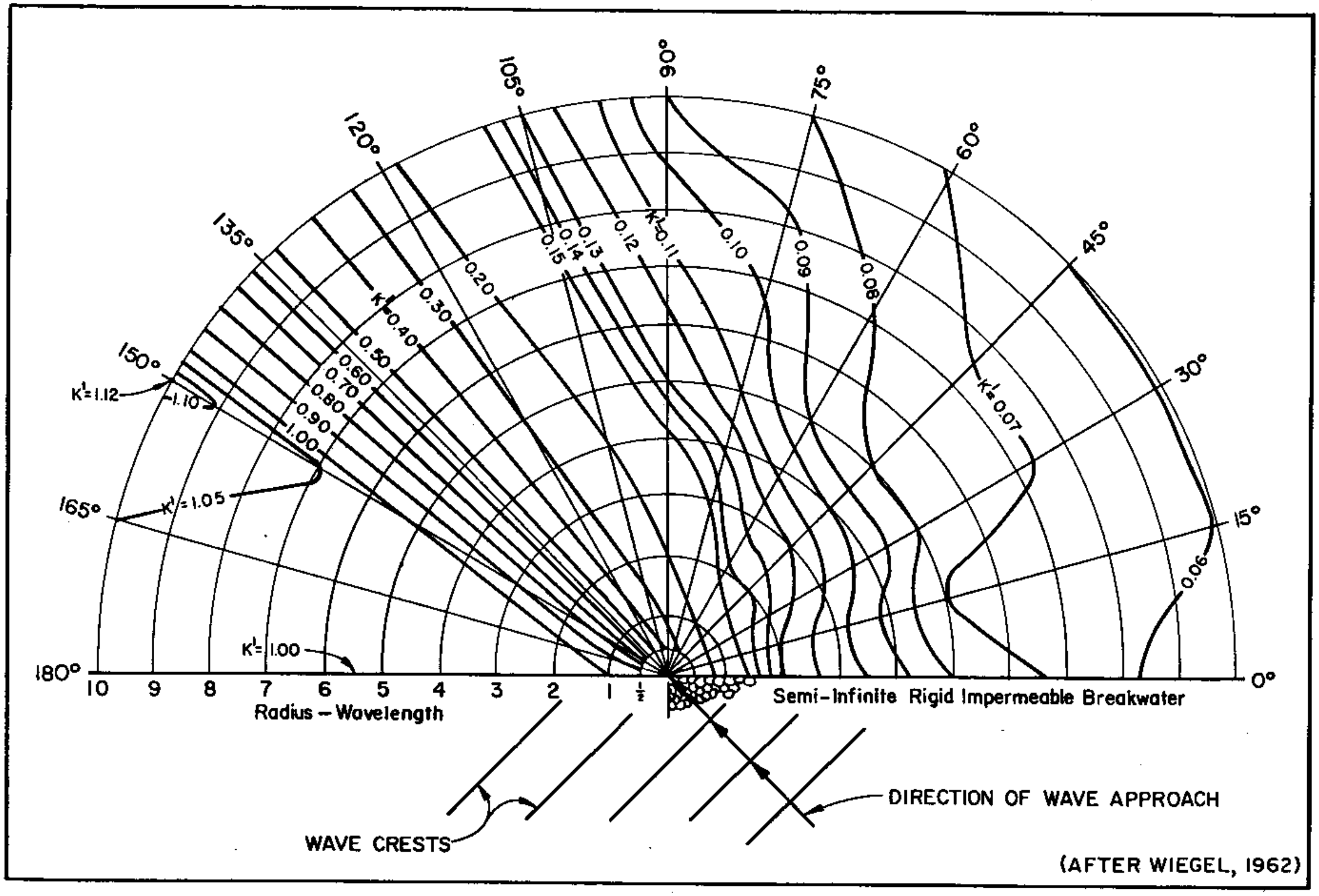


FIGURE 18
Wave-Diffraction Diagram for 135° Angle of Wave Approach

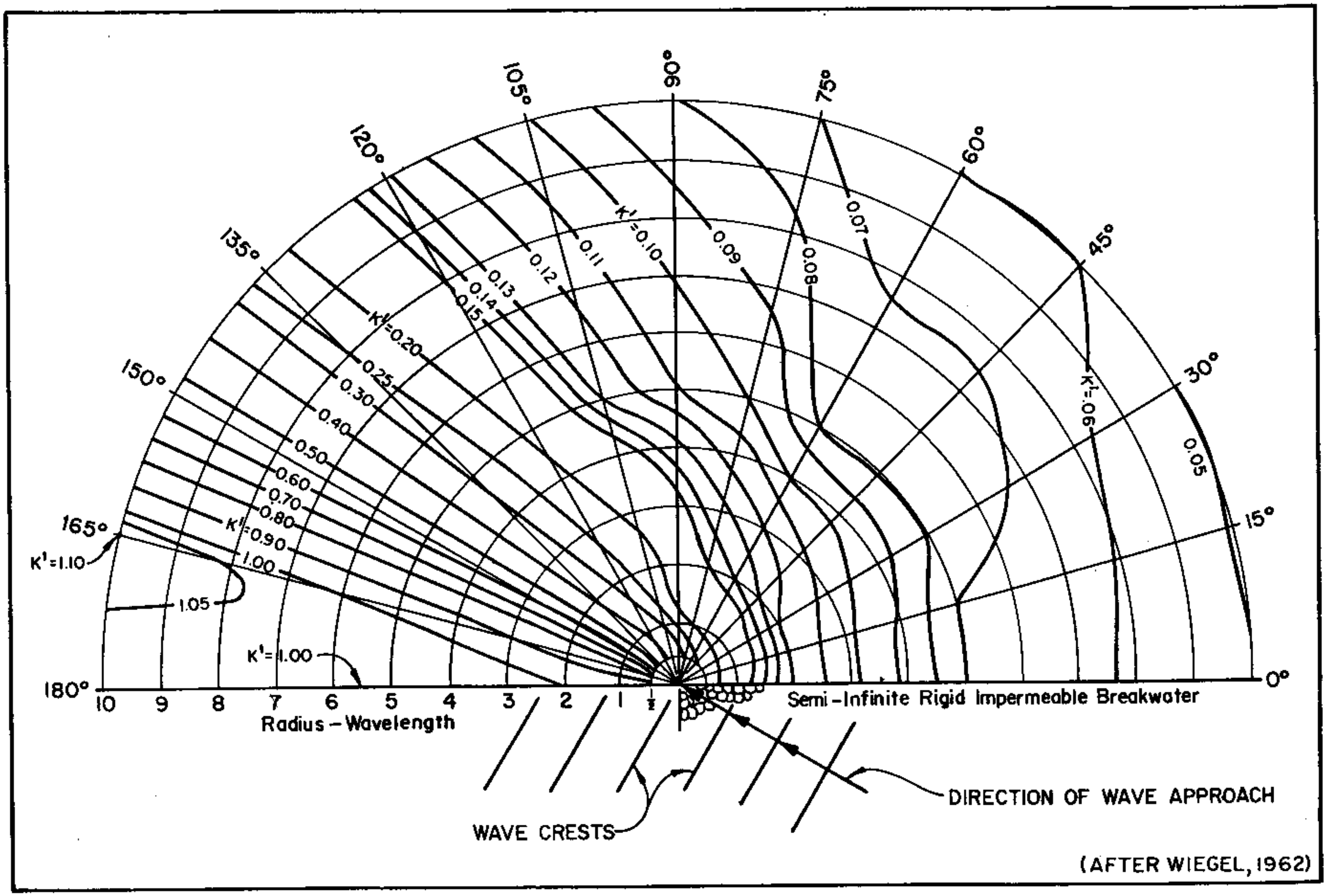


FIGURE 19
Wave-Diffraction Diagram for 150° Angle of Wave Approach

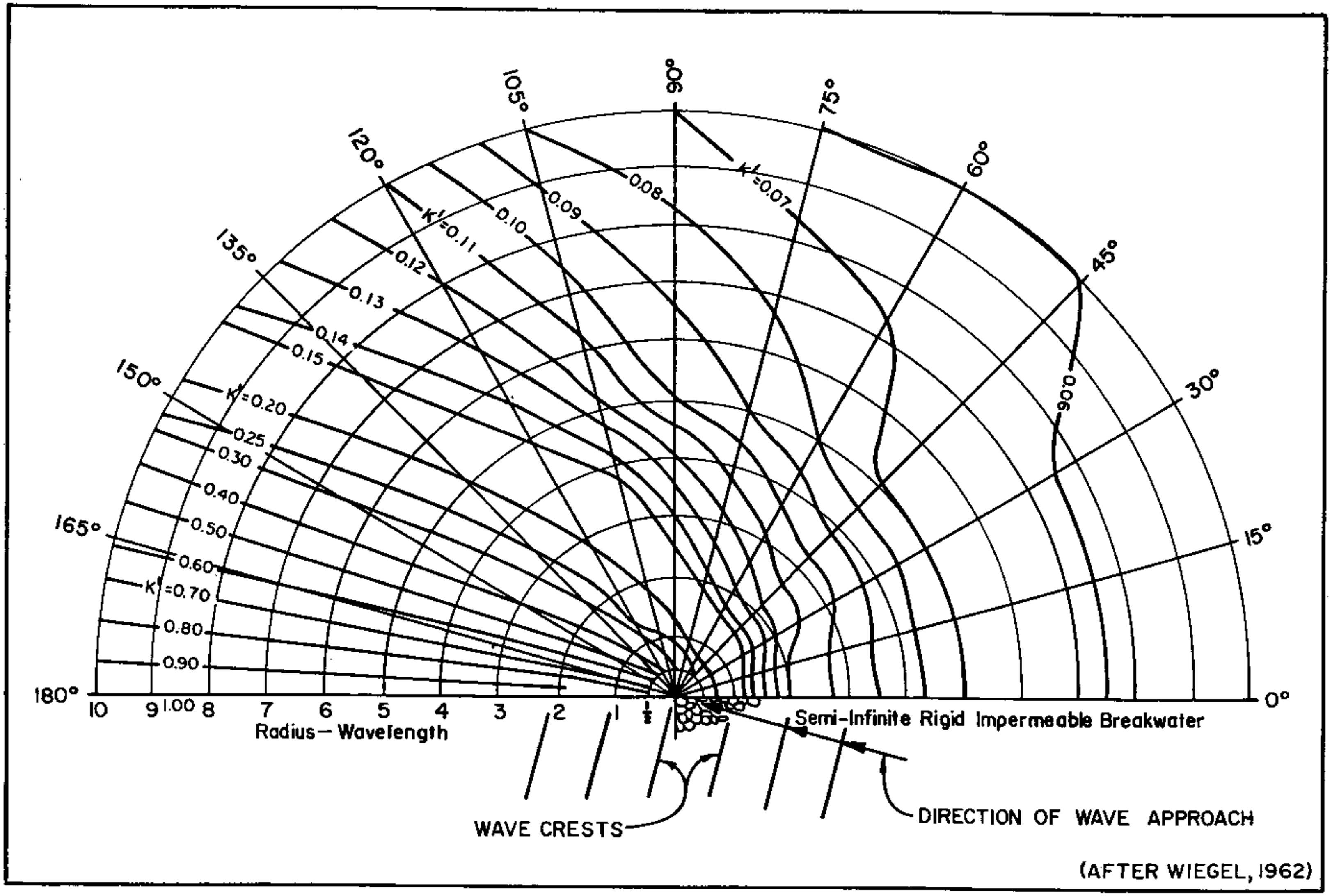


FIGURE 20
Wave-Diffraction Diagram for 165° Angle of Wave Approach

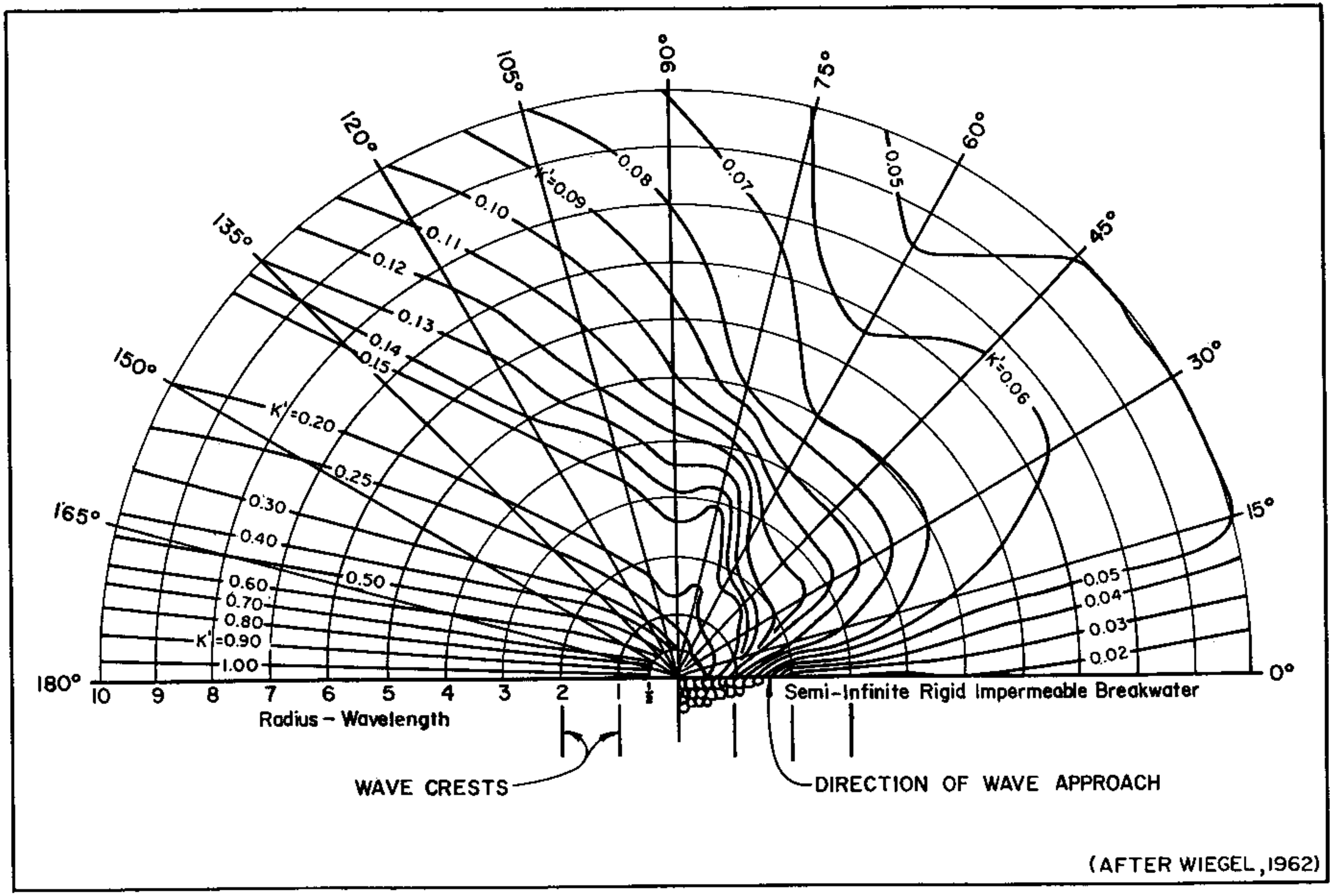


FIGURE 21
Wave-Diffraction Diagram for 180° Angle of Wave Approach

Diffraction diagrams are constructed in polar-coordinate form and consist of arcs spaced one "radius-wavelength unit" apart, and rays, spaced 15 deg. apart. These arcs and rays are centered at the intersection of the breakwater head with the stillwater level. The diagrams in Figures 10 through 21 show the breakwater extending to the right when looking toward the area of diffraction. (These diagrams are used for a breakwater extending to the left by simply turning over the diagrams to their opposite sides.) The angle of wave approach is measured counterclockwise from the breakwater. (This angle would be measured clockwise for a breakwater extending to the left.) To adjust a given diffraction diagram to the scale of a given working drawing, the diagram must be scaled up or down so that one radius-wavelength unit on the diffraction diagram is equal to one wavelength on the working drawing. A template overlay of the scaled diffraction diagram is then prepared; thus, lines of constant K' (isolines) can be easily transferred to the working drawing.

An example of the use of diffraction diagrams would be to determine the breakwater length needed to protect a boat basin. Breakwater length is measured along the breakwater on the diffraction diagram in terms of radius-wavelength units, which are then converted to feet, using the map scale, to determine design breakwater length needed to achieve a given K' in a given region in the breakwater's lee. This procedure is outlined in Example Problem 6.

In the use of diffraction diagrams, wave-crest lines are required to estimate the combined effects of refraction and diffraction. Wave crests may be approximated with sufficient accuracy by circular arcs. For a single breakwater, the arcs will be centered at the intersection of the breakwater head with the still water level. That part of the wave crest extending into unprotected water beyond the K' = 0.5 line may be approximated by a straight line. Caution should be exercised because diffraction diagrams assume a constant water depth and assume that the breakwater is thin compared with the wavelength. Refraction effects should be taken into account over rapidly varying bottom depths. As a general rule, diffraction predominates over the first three wavelengths; then, if the bottom varies rapidly, refraction should be considered.

EXAMPLE PROBLEM 6

Given: a. Incident wave: HÚi¿ = 5 feet
T = 10 seconds
[phi] = 75 deg.

b. Depth at breakwater toe, dÚs¿ = 20 feet

c. See Figure 22 for map of boat basin layout; scale on this layout is 1 inch = 600 feet.

Find: Length of breakwater required to protect a boat basin by maintaining wave heights at less than 1.5 feet in the berthing area.

Solution: (1) Find the wavelength, L, at the toe of the breakwater head, where dÚs¿ = 20 feet:

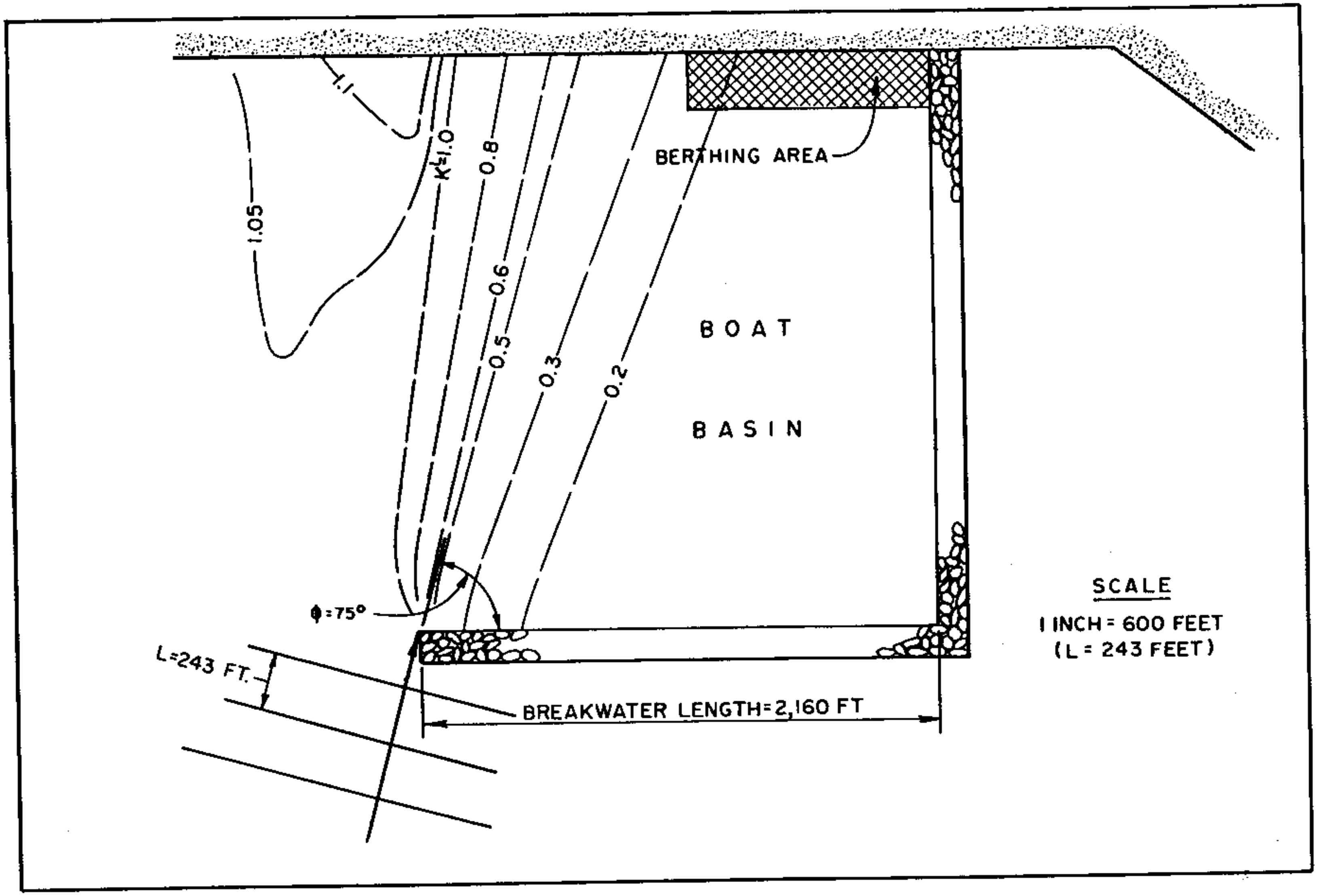


FIGURE 22
Boat Basin Layout and Diffraction Diagram for Example Problem 6

EXAMPLE PROBLEM 6 (Continued)

$L = (g/2[pi])\ T^2 = (32.2/2[pi])(10)^2 = 512$ feet

$d_s/L_o = 20/512 = 0.0391$

From Figure 2 for $d_s/L_o = 0.0391$:

$d_s/L = 0.0822$

THEREFORE: $L = 20/0.0822 = 243$ feet

(2) The scale of the basin layout map is 1:600; that is, 1 inch = 600 feet. Therefore, the wavelength, L = 243 feet, is 0.4 inches on the map. This 0.4 inches represents one radius-wavelength unit.

The diffraction diagram is scaled so that one radius-wavelength unit on the diagram is equal to one wavelength (0.4 inches) on the map.

Figure 14 is the diffraction diagram used ([phi] = 75 deg.). An overlay of the scaled Figure 14 was prepared and laid over the basin layout to produce Figure 22.

(3) Desired $K' = H/H_i = 1.5/5 = 0.30$. The K' = 0.30 line of the overlay should not intersect the berthing area. The length of the breakwater required to keep the K' = 0.30 line from intersecting the berthing area is thus determined to be nine radius-wavelength units.

1 radius-wavelength unit = 0.4 inches

THEREFORE: 9 radius-wavelength units = (9)(0.4) = 3.6 inches

Map scale is 1 inch = 600 feet.

THEREFORE: 3.6 inches = (3.6)(600) = 2,160 feet

Therefore, the required breakwater length is 2,160 feet.

(2) Gap Width Less Than Five Wavelengths at Normal Incidence. The determination of diffraction when the breakwater-gap width, B, is less than five wavelengths is more complex than that for a single, semi-infinite breakwater. A separate diagram must be drawn for each ratio of gap width to wavelength, B/L. The diagram for a B/L ratio of 2, shown in Figure 23, illustrates a symmetrical diagram, with the wave crests drawn on it for the purpose of illustrating its use. Figures 24 through 33 show lines of equal diffraction coefficients for B/L ratios of 0.50, 1.00, 1.41, 1.64, 1.78, 2.00, 2.50, 2.95, 3.82, and 5.00, respectively. Unlike Figure 23, only one-half of the diffraction diagram is presented on each figure of Figures 24-33; the diagrams are symmetrical about the line x/L = 0.

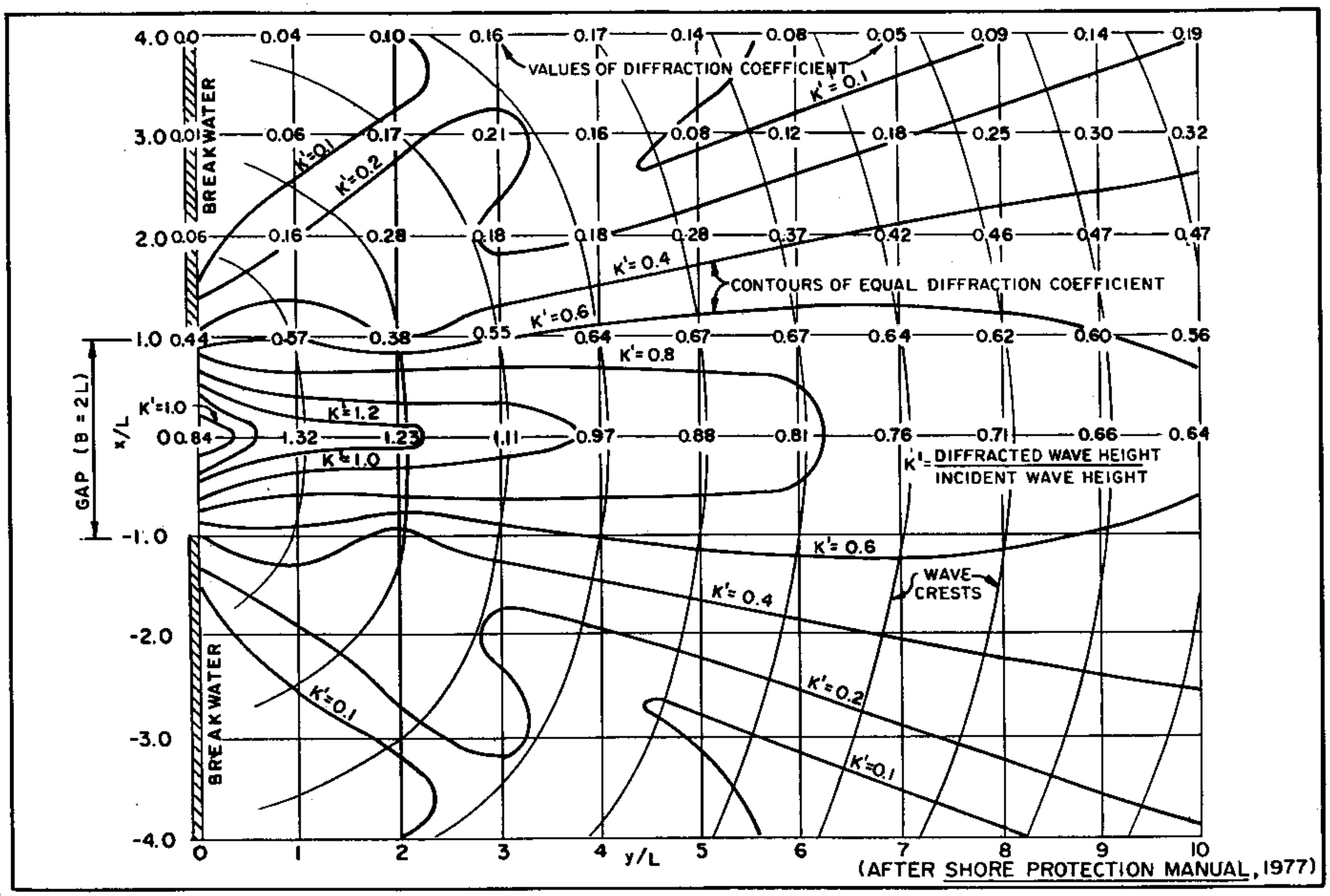


FIGURE 23
Generalized Diffraction Diagram for Two Breakwaters for B = 2.00 L (B/L = 2.00)

$B = 2.00\ L\ (B/L = 2.00)]$

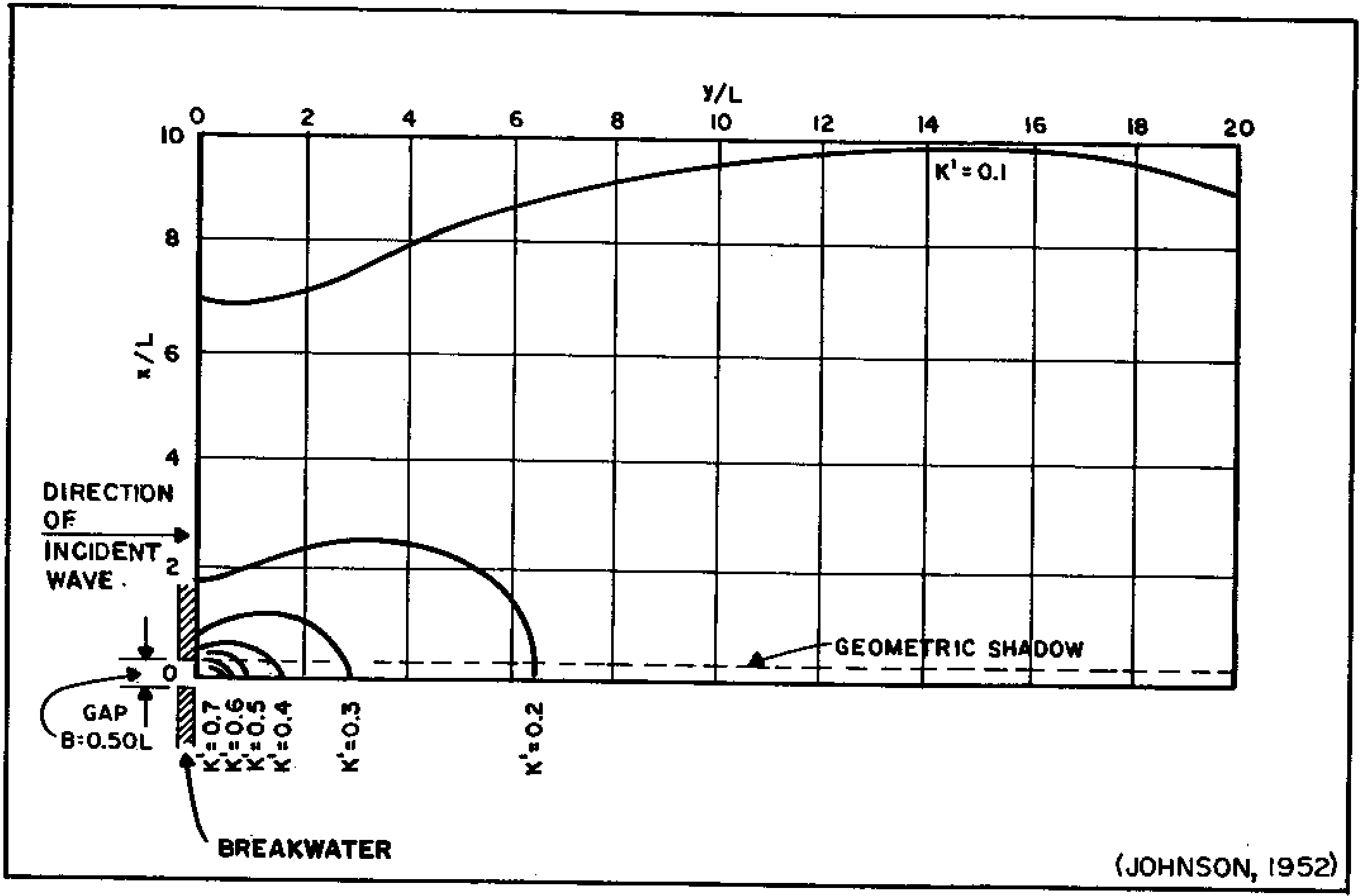


FIGURE 24
Contours of Equal Diffraction Coefficient for B = 0.50 L (B/L = 0.50)

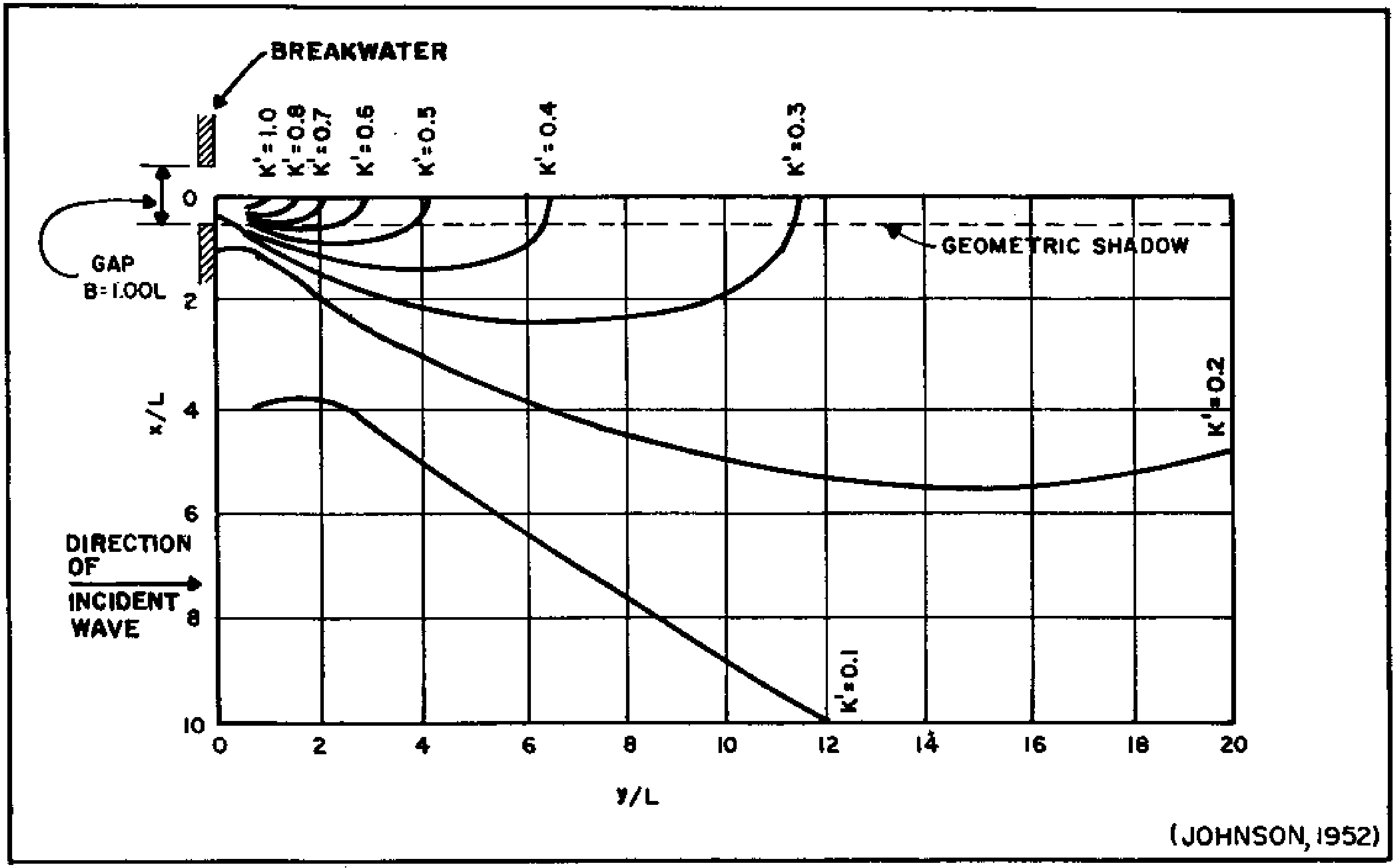


FIGURE 25
Contours of Equal Diffraction Coefficient for B = 1.00 L (B/L = 1.00)

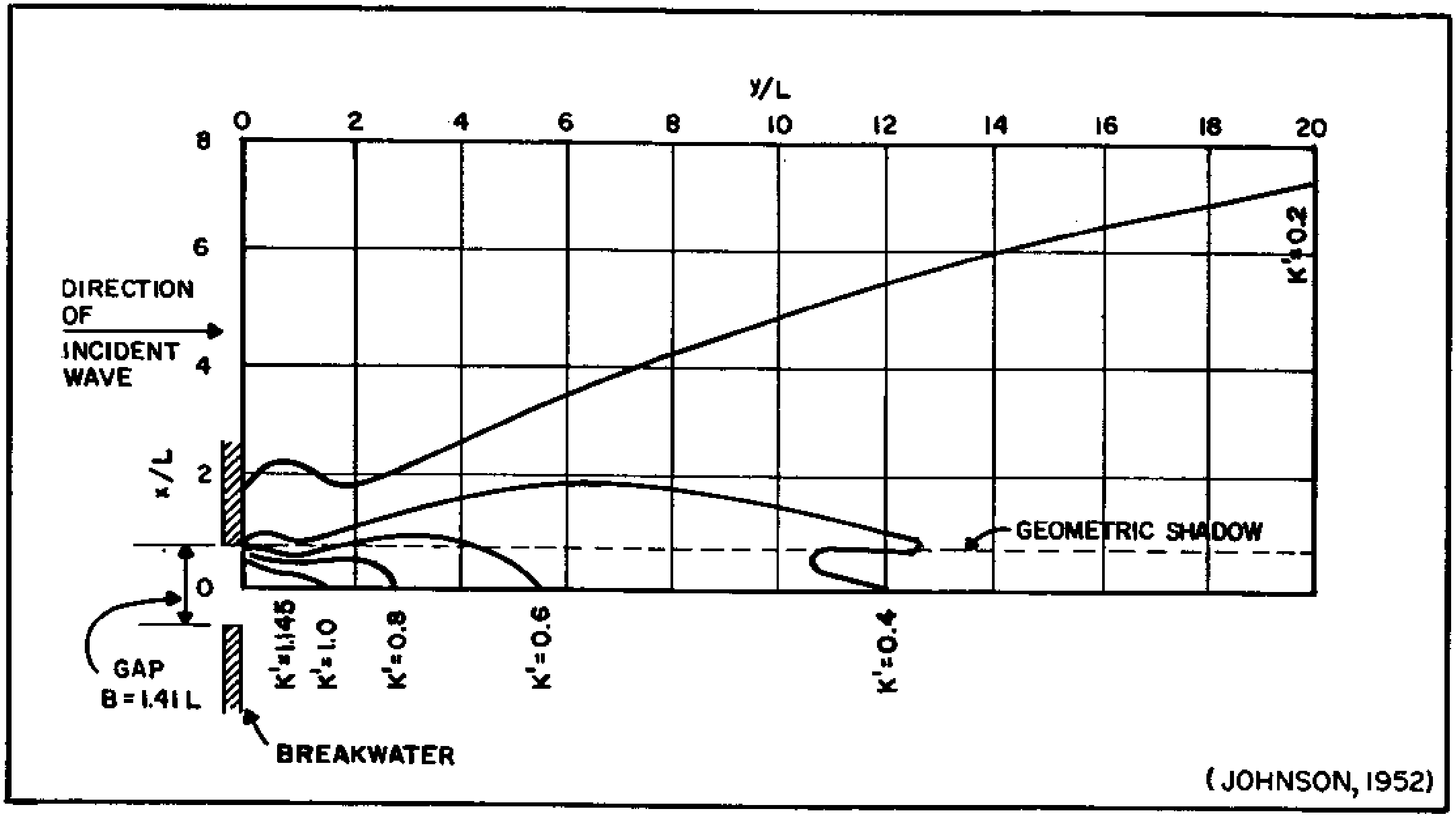


FIGURE 26
Contours of Equal Diffraction Coefficient for B = 1.41 L (B/L = 1.41)

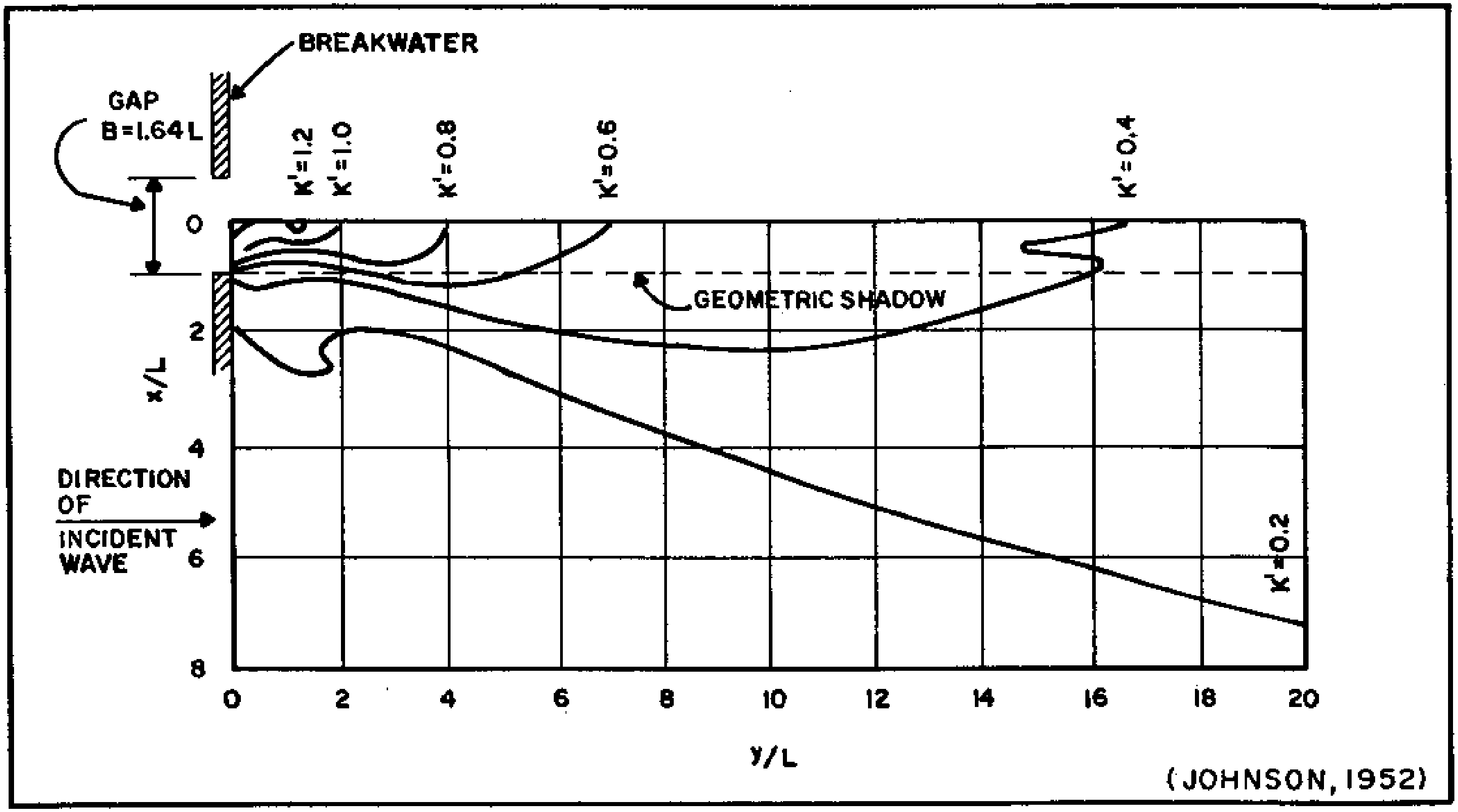


FIGURE 27
Contours of Equal Diffraction Coefficient for B = 1.64 L (B/L = 1.64)

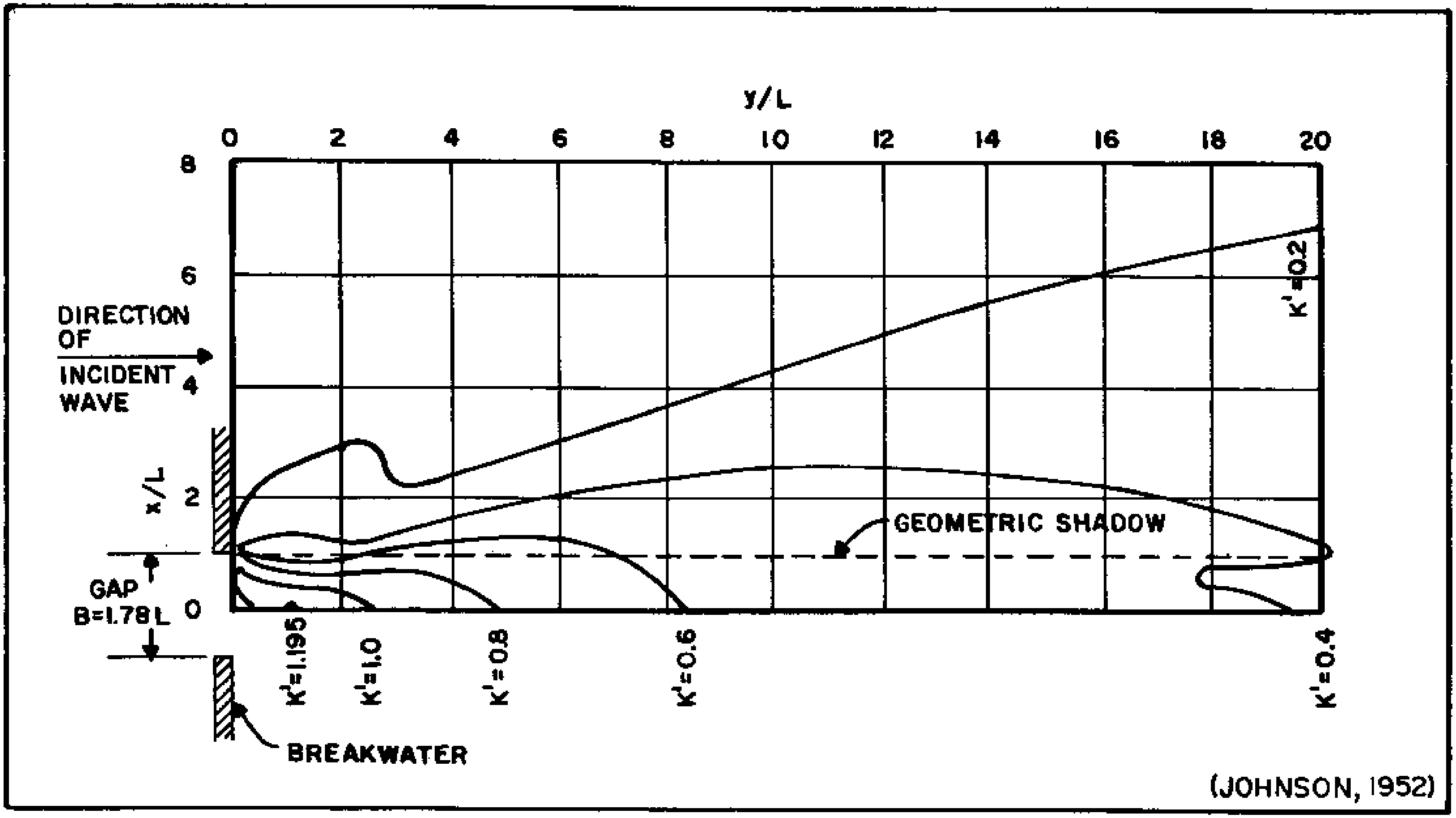


FIGURE 28
Contours of Equal Diffraction Coefficient for B = 1.78 L (B/L = 1.78)

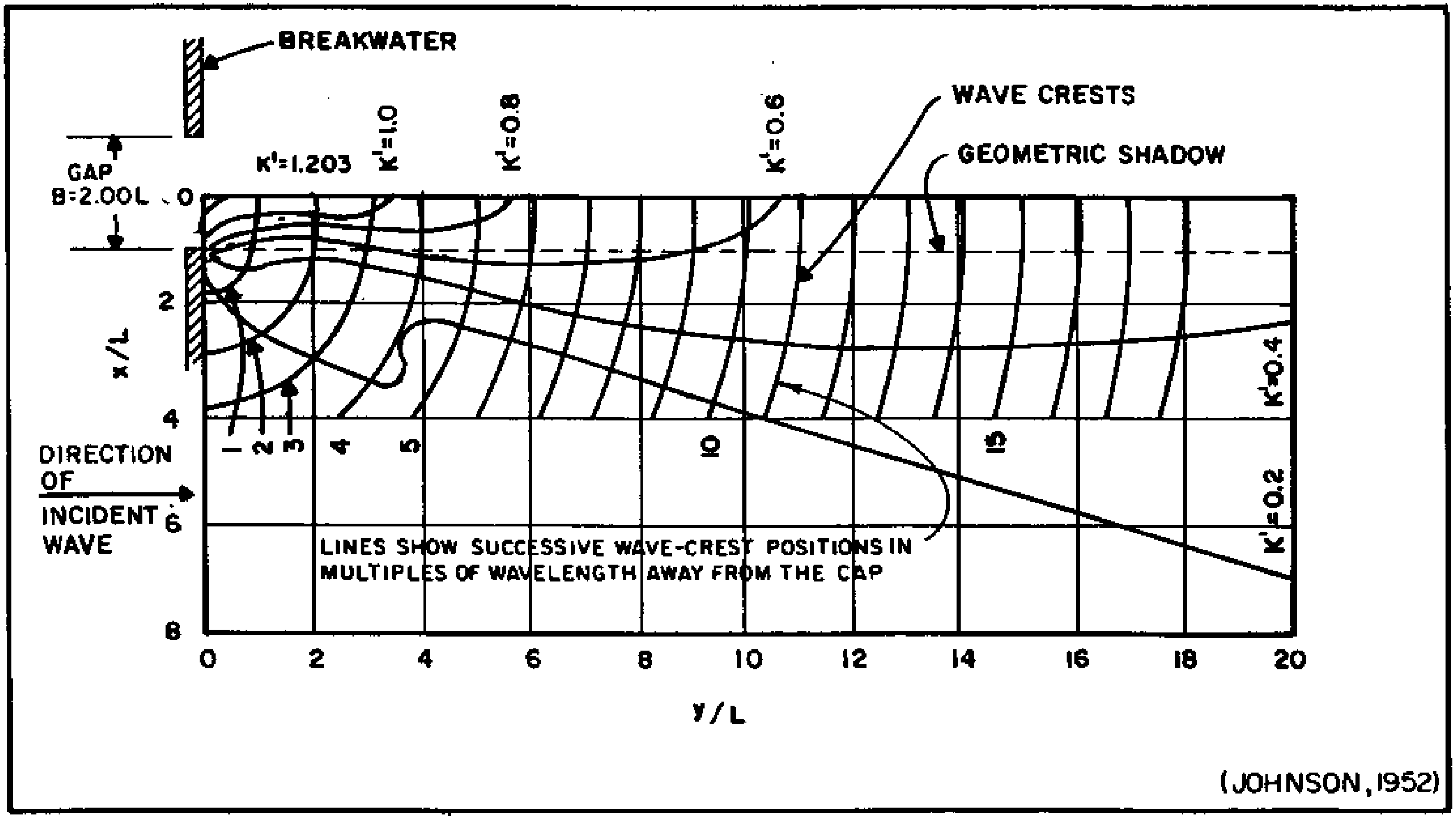


FIGURE 29
Contours of Equal Diffraction Coefficient for B = 2.00 L (B/L = 2.00)

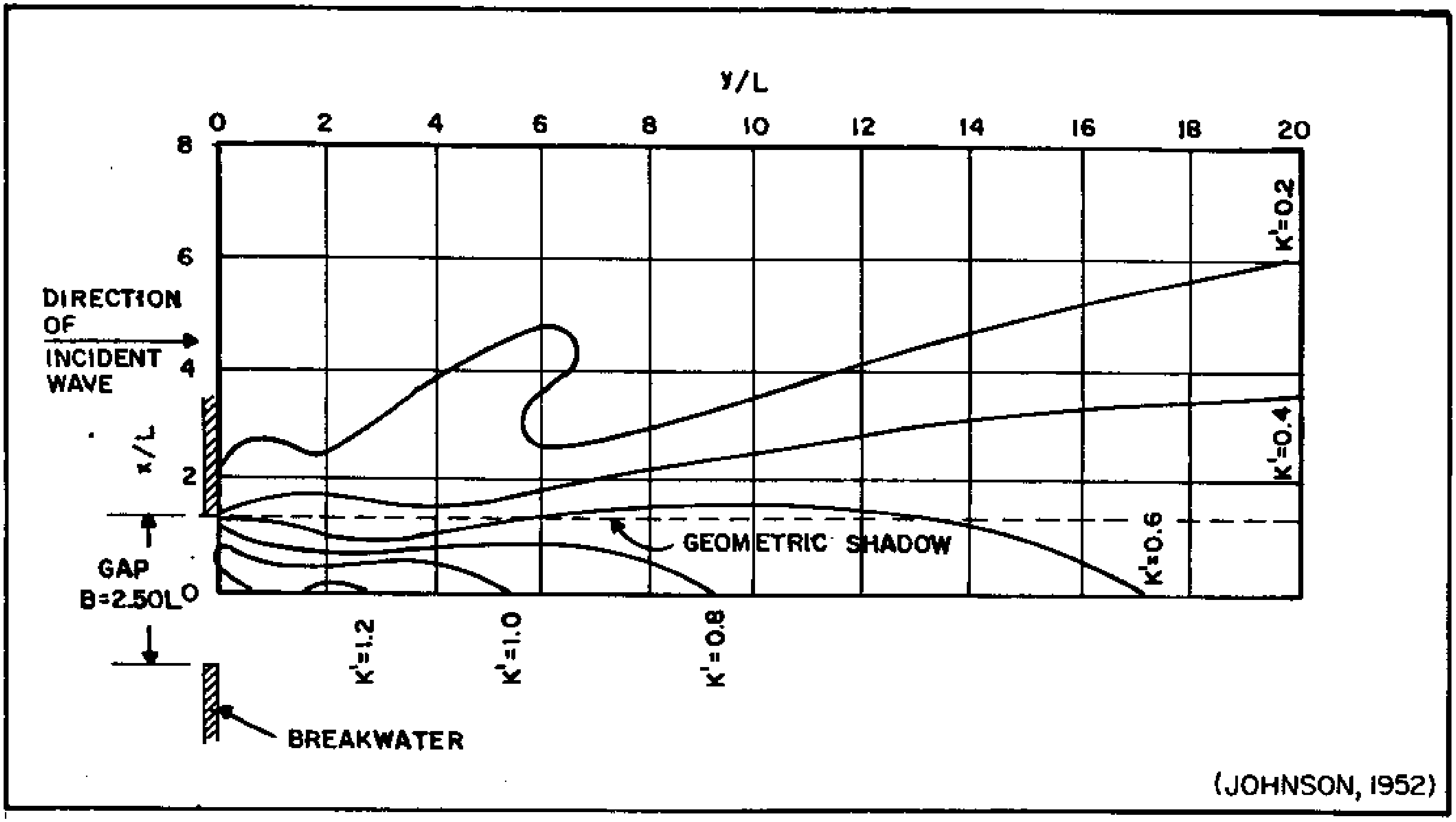


FIGURE 30
Contours of Equal Diffraction Coefficient for B = 2.50 L (B/L = 2.50)

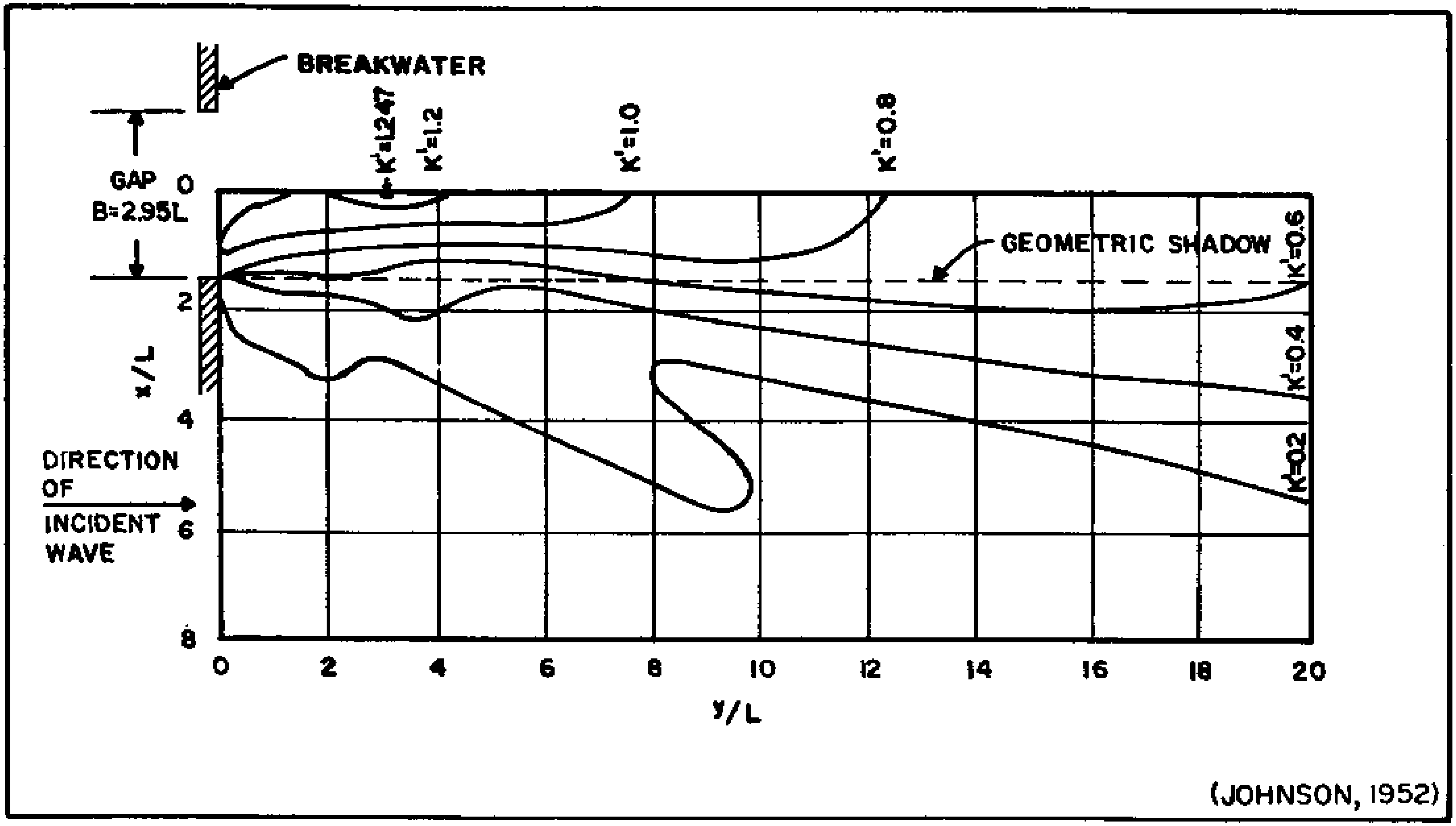


FIGURE 31
Contours of Equal Diffraction Coefficient for B = 2.95 L (B/L = 2.95)

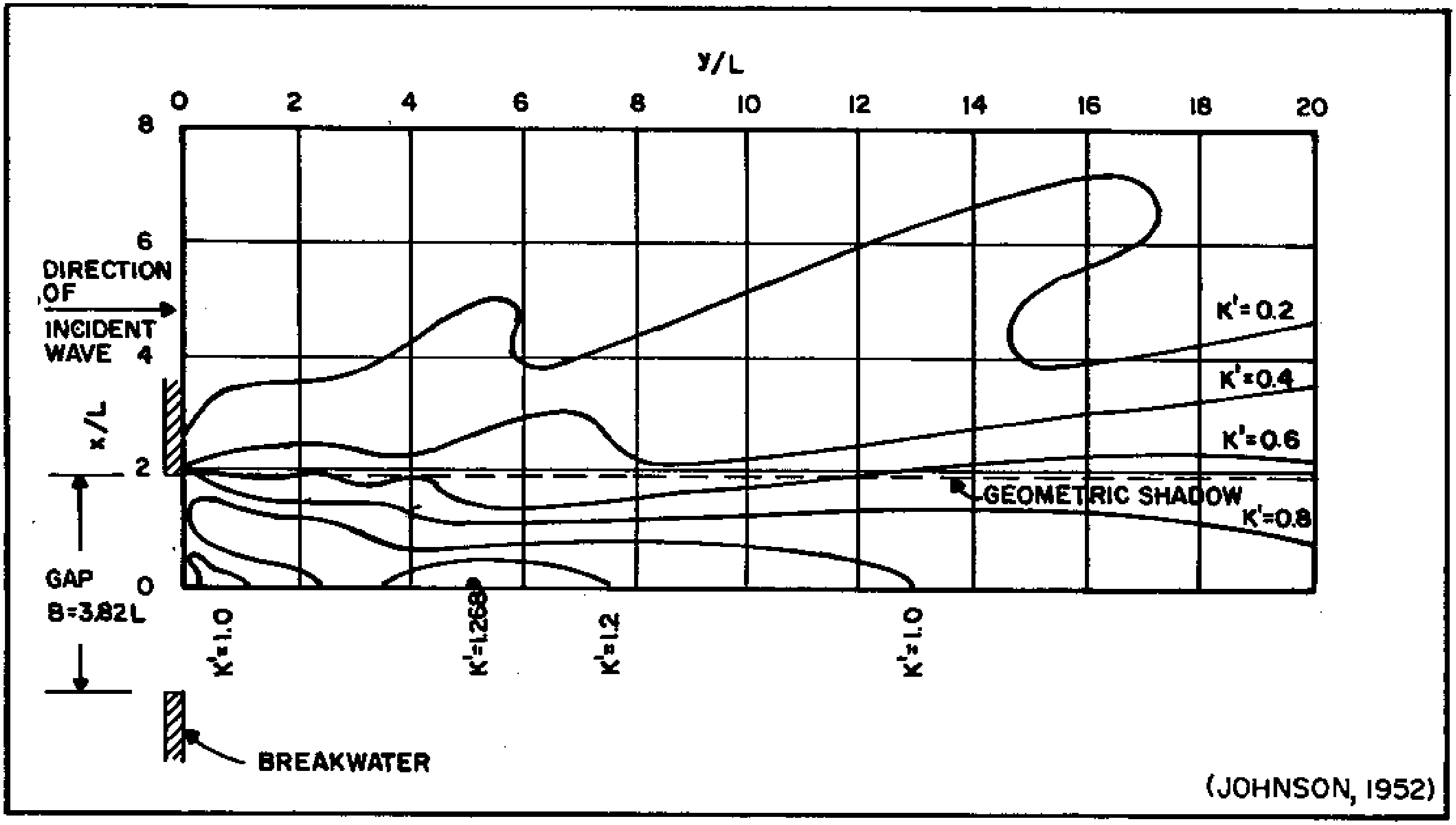


FIGURE 32
Contours of Equal Diffraction Coefficient for B = 3.82 L (B/L = 3.82)

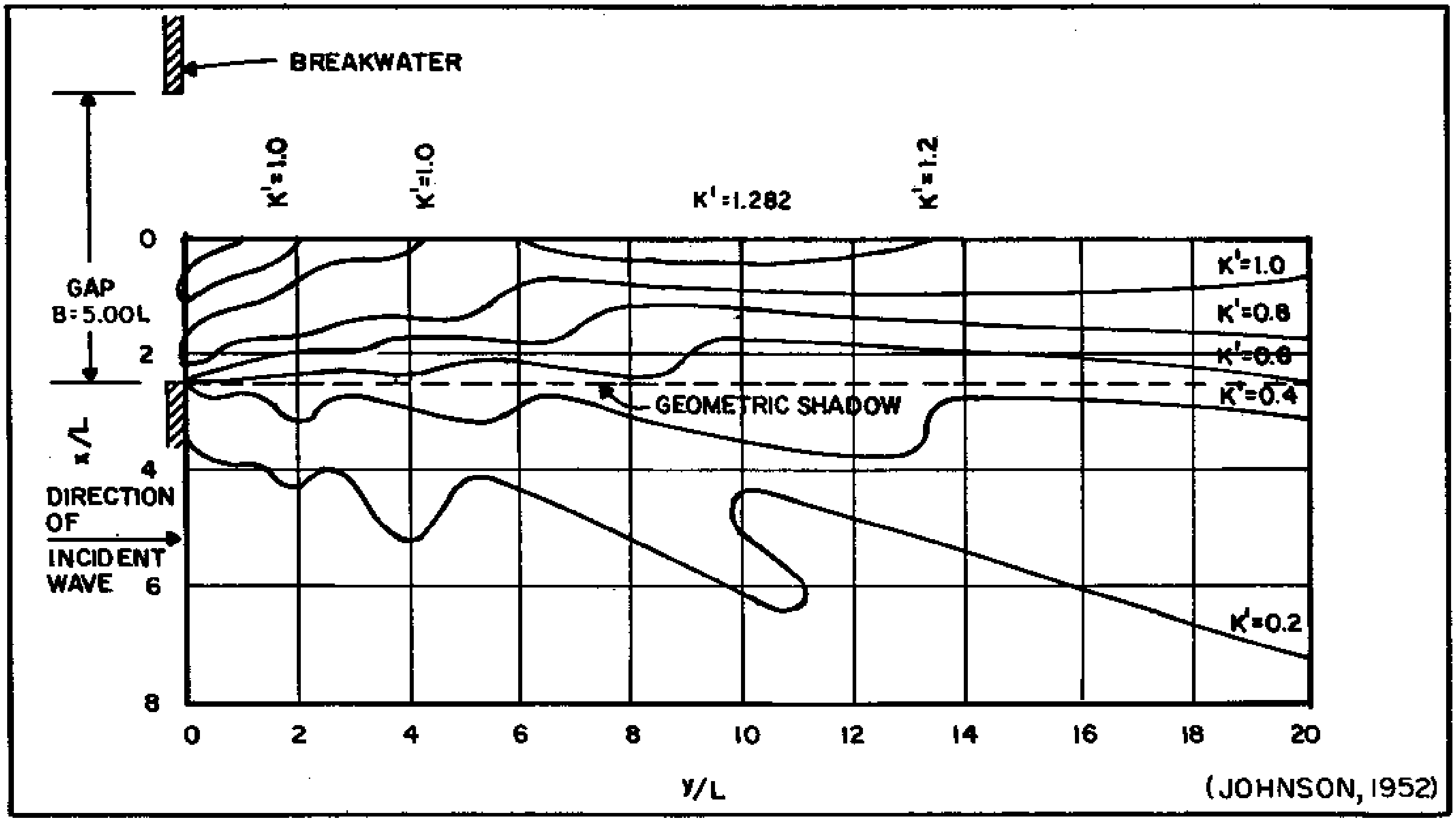


FIGURE 33
Contours of Equal Diffraction Coefficient for B = 5.00 L (B/L = 5.00)

Wave crests to about six wavelengths may be approximated by two arcs centered on the head of each breakwater and connected by a smooth curve (approximated by a circular arc entered at the middle of the gap). Crests that are more than eight wavelengths behind the breakwater may be approximated by an arc centered at the middle of the gap.

(3) Gap Width Greater Than Five Wavelengths at Normal Incidence. Where the breakwater-gap width is greater than five wavelengths, the diffraction effects about each breakwater are nearly independent. The diagram (see Figure 15) for a single breakwater with a 90 deg. wave-approach angle may be used to define the diffraction characteristics in the lee of both breakwaters. (See Figure 34.)

(4) Gap at Oblique Incidence. When waves approach at an angle to the axis of a breakwater, the diffracted wave characteristics differ from those resulting when waves approach normal to the axis. An approximate determination of diffracted wave characteristics may be obtained by considering the gap to be as wide as its projection in the direction of incident wave travel, B', as shown in Figure 35. Calculated diffraction diagrams for wave-approach angles of 0 deg., 15 deg., 30 deg., 45 deg., 60 deg., and 75 deg. are shown in Figures 36 through 41, respectively. Use of these diagrams will give more accurate results than the approximation method.

d. Wave Decay. Waves leaving their generating area radiate energy laterally through angular spreading. Waves also decay by viscous dissipation as they propagate out of their generating area. A general rule of thumb is that a wave loses one-third of its height when the distance in nautical miles it travels equals the wavelength in feet. Thus, short-period waves die out more rapidly than longer-period waves. Other factors, such as winds, currents, and other wave systems, modify waves that propagate out of their generating area. Waves propagating over shallow water decay by bottom friction and percolation. (Refer to Ippen (1966) for determination of decay due to bottom friction and percolation.) Only in special cases are reductions due to bottom friction and percolation used in design practice. Neglect of these energy-reducing factors should lead to a conservative design. Bottom dissipation effects are accounted for in Subsection 3, WAVE HINDCASTING, in Section 2.

e. Wave Breaking.

(1) Limiting Factors. Waves become unstable and break when either the wave steepness, H/L, is > / = 0.142, or the wave height relative to the water depth, H/d, is on the order of unity.

(2) Depth of Water at Breaking and Breaking-Wave Height. The depth of water at breaking, d_b, and the breaking-wave height, H_b, are functions of the bottom slope, m, an the wave steepness, H/L. The relative breaker 2 height, H_b/H_o, can be found from Figure 42 for the given values of $H'_o/g T^2$ and slope, m. (Where the bottom slope varies, choose a representative composite slope for about one wavelength seaward of the breaking point.) The relative breaker depth, d_b/H_b, can then be determined from Figure 43 for the appropriate $H_b/g T^2$ and slope, m. For a flat bottom, the breaking-wave height, H_b, is equal to 0.78 d_b.

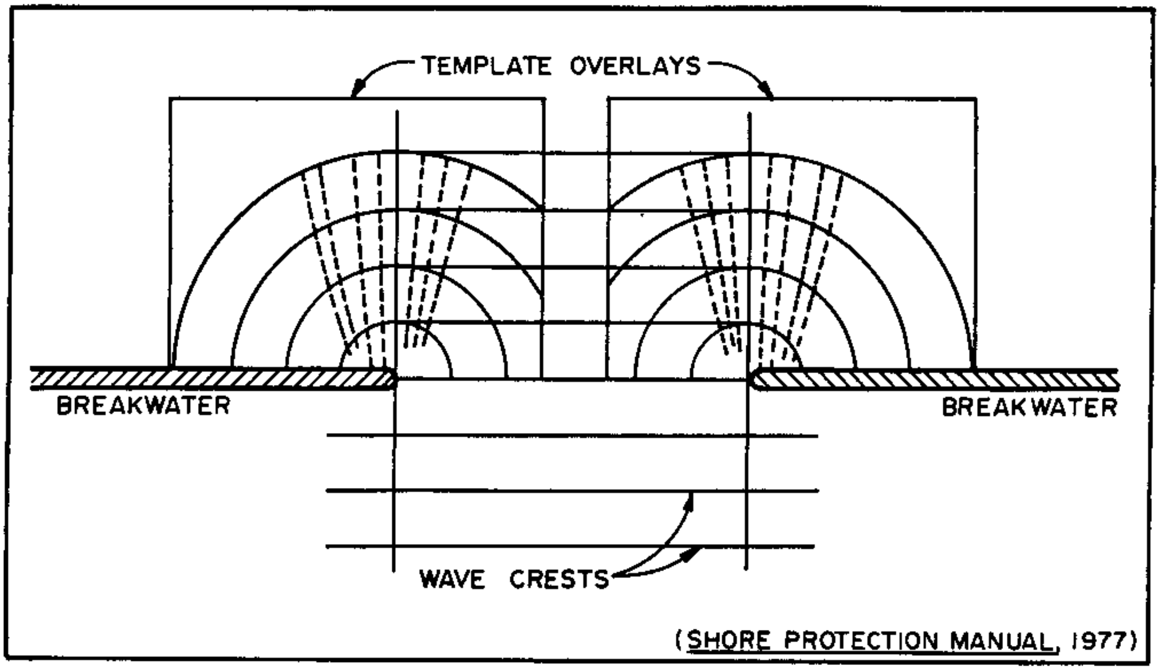


FIGURE 34
Diffraction Diagram for B > 5.00 L and Normal Incidence

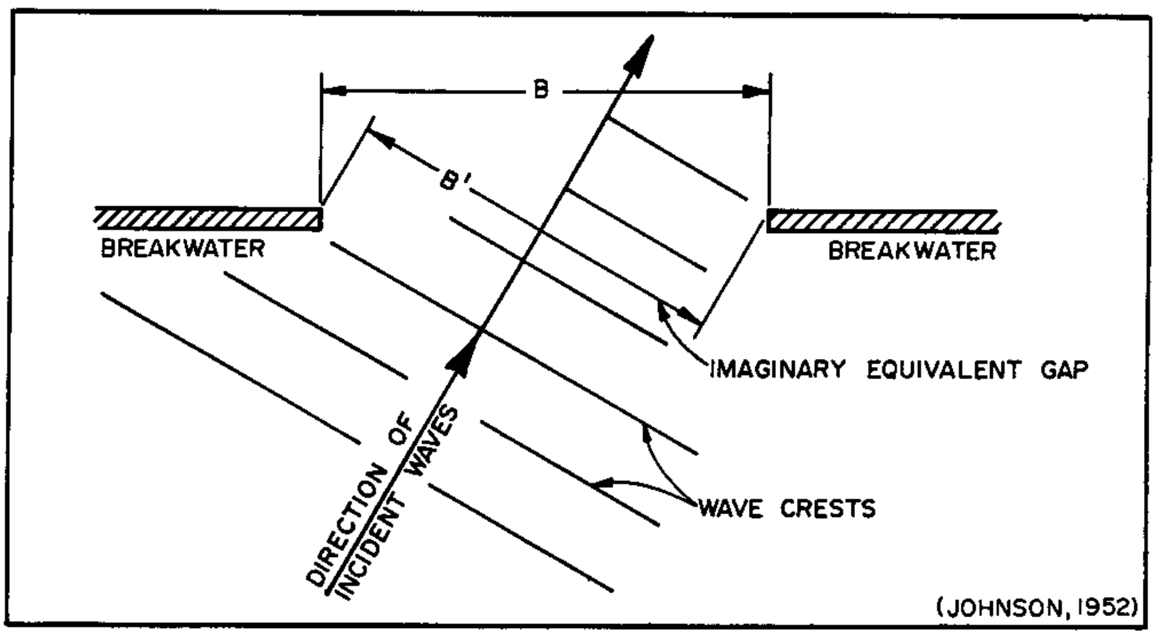


FIGURE 35
Gap at Oblique Incidence Considered to be as Wide as Imaginary Equivalent Gap

Incidence and Gap at Oblique Incidence
Considered to be as Wide as Imaginary Equivalent

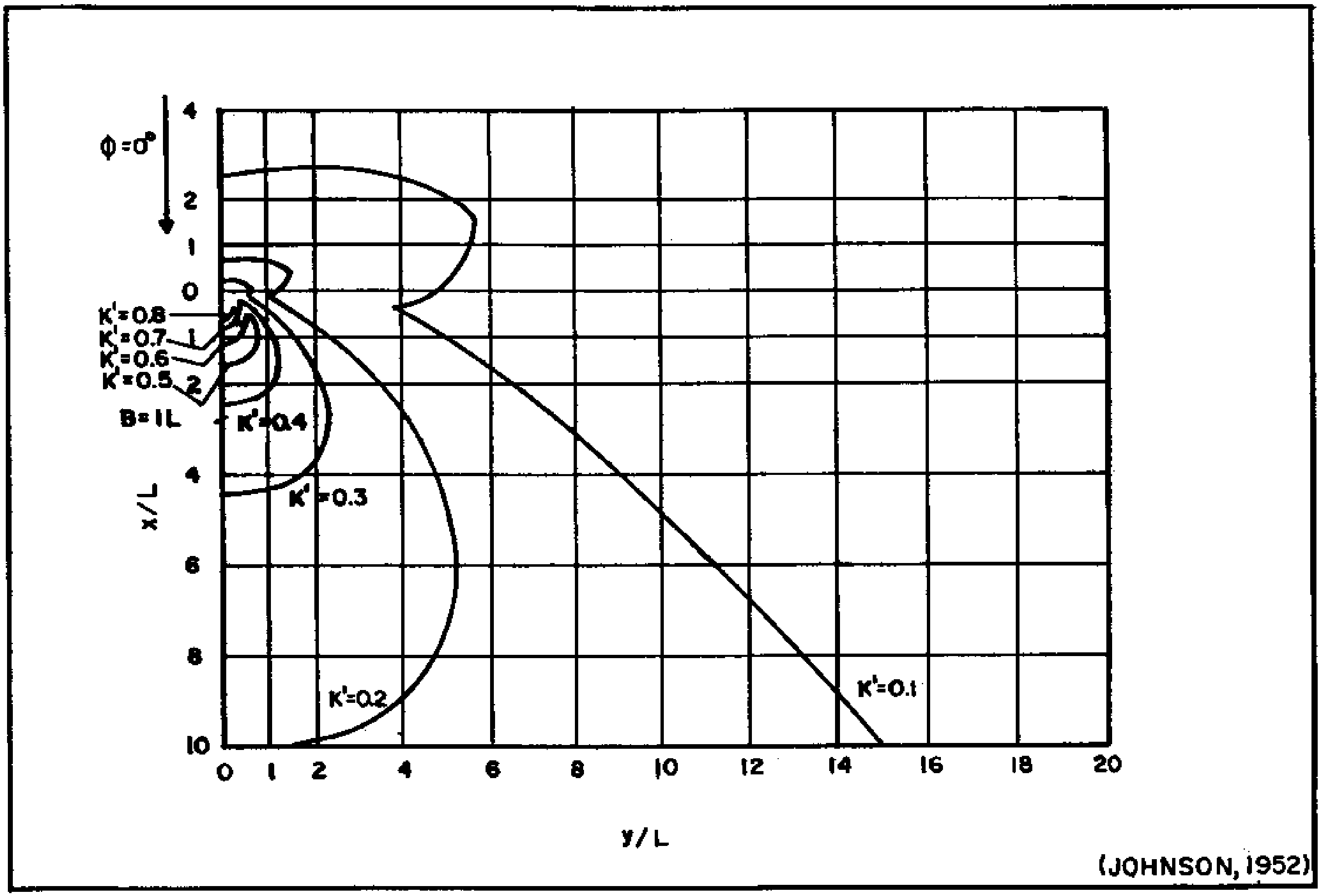


FIGURE 36
Diffraction for a Breakwater Gap of One Wavelength Width and $\phi = 0°$

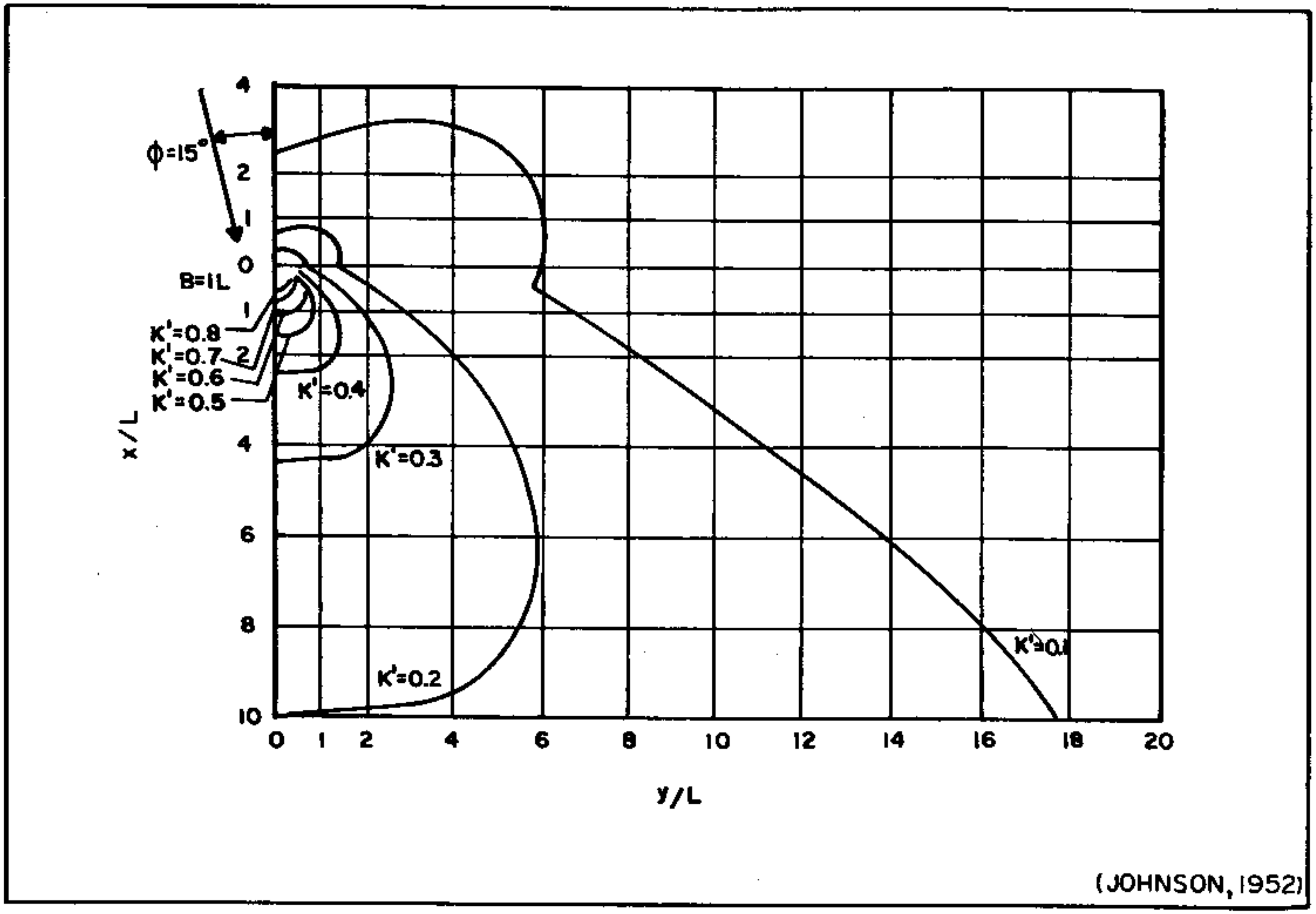


FIGURE 37
Diffraction for a Breakwater Gap of One Wavelength Width and $\Phi = 15°$

Width and [phi] = 15 deg.]

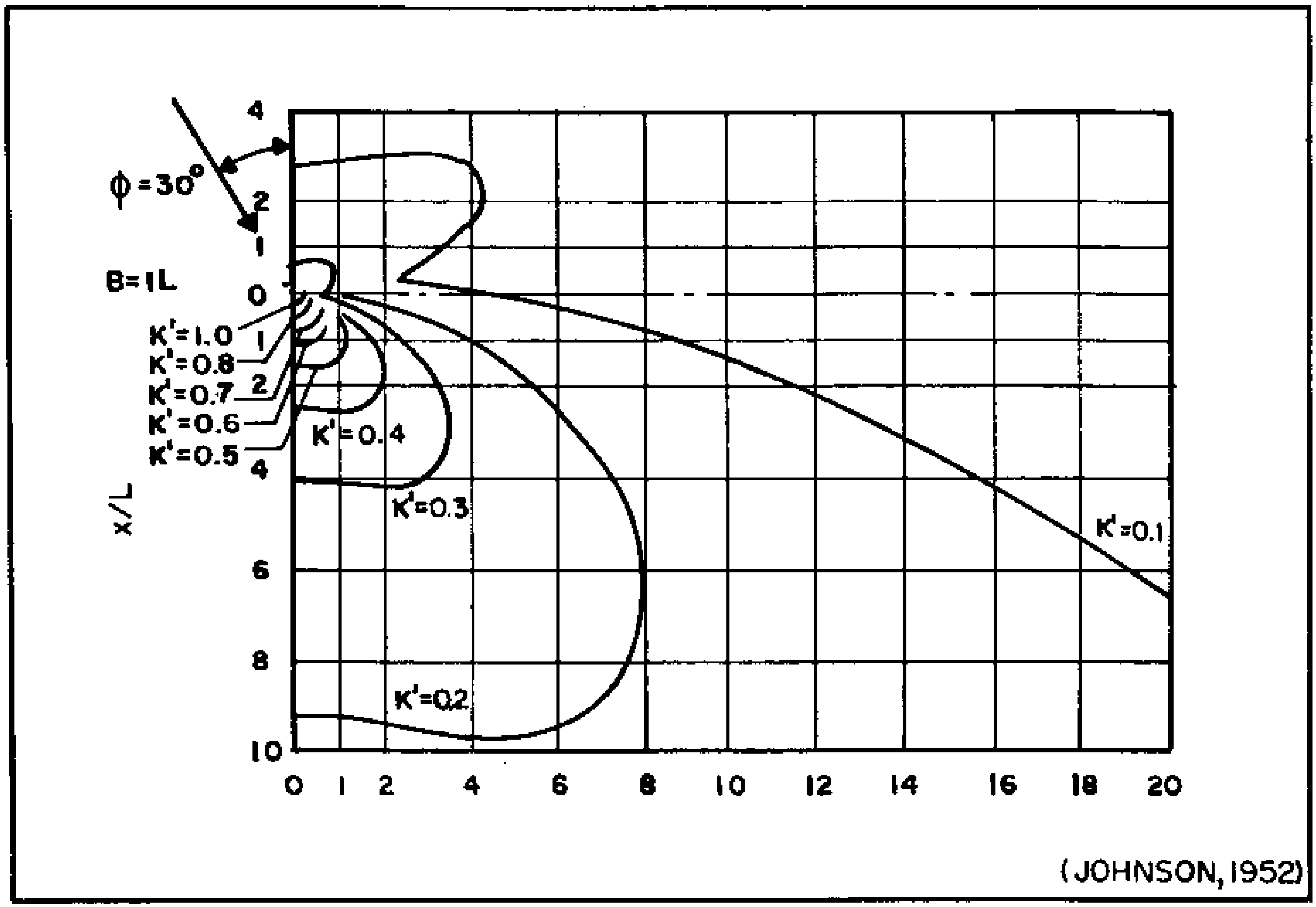


FIGURE 38
Diffraction for a Breakwater Gap of One Wavelength Width and $\phi = 30°$

Width and [phi] = 30 deg.]

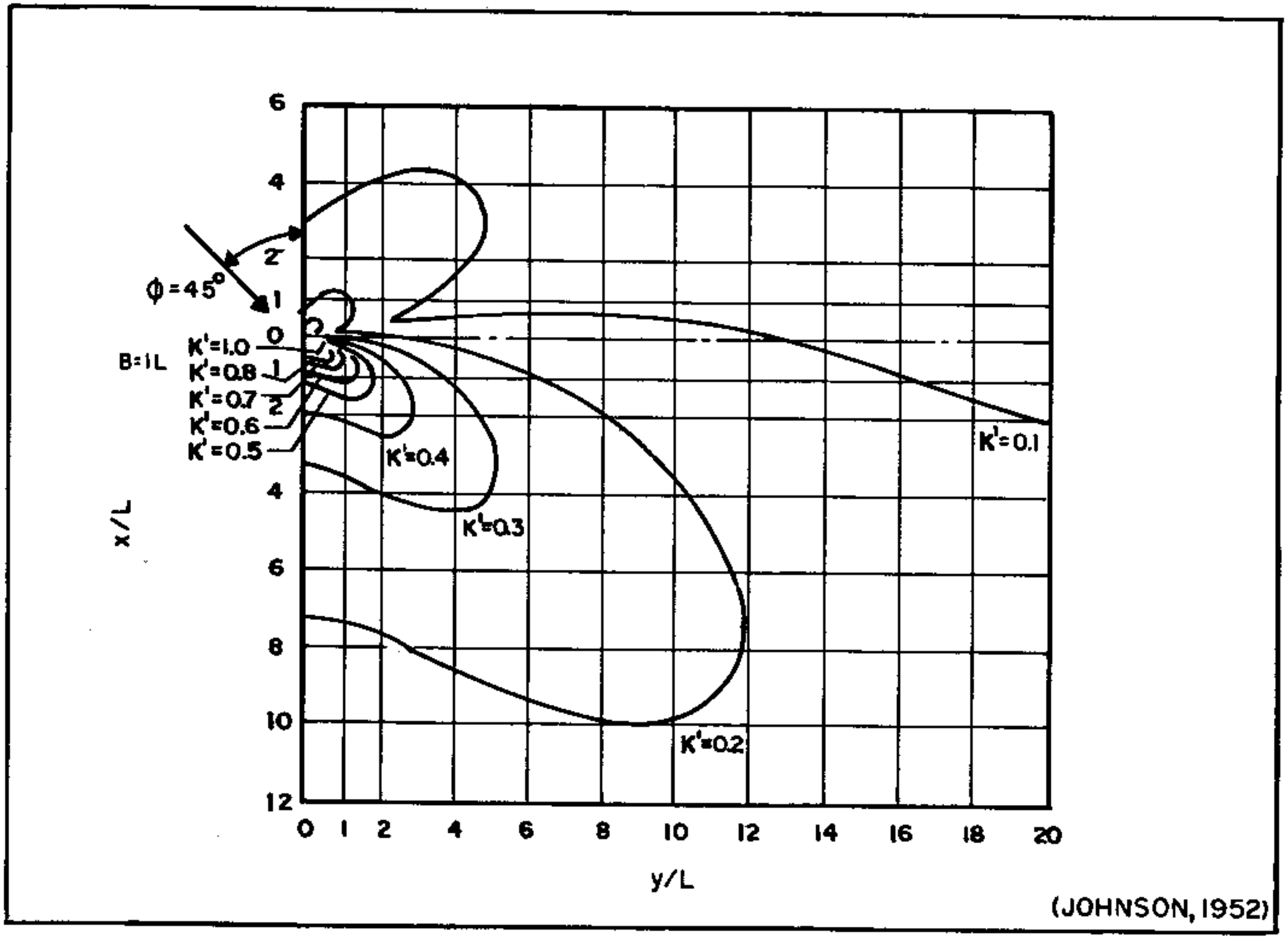


FIGURE 39
Diffraction for a Breakwater Gap of One Wavelength Width and $\phi = 45°$

Width and [phi] = 45 deg.]

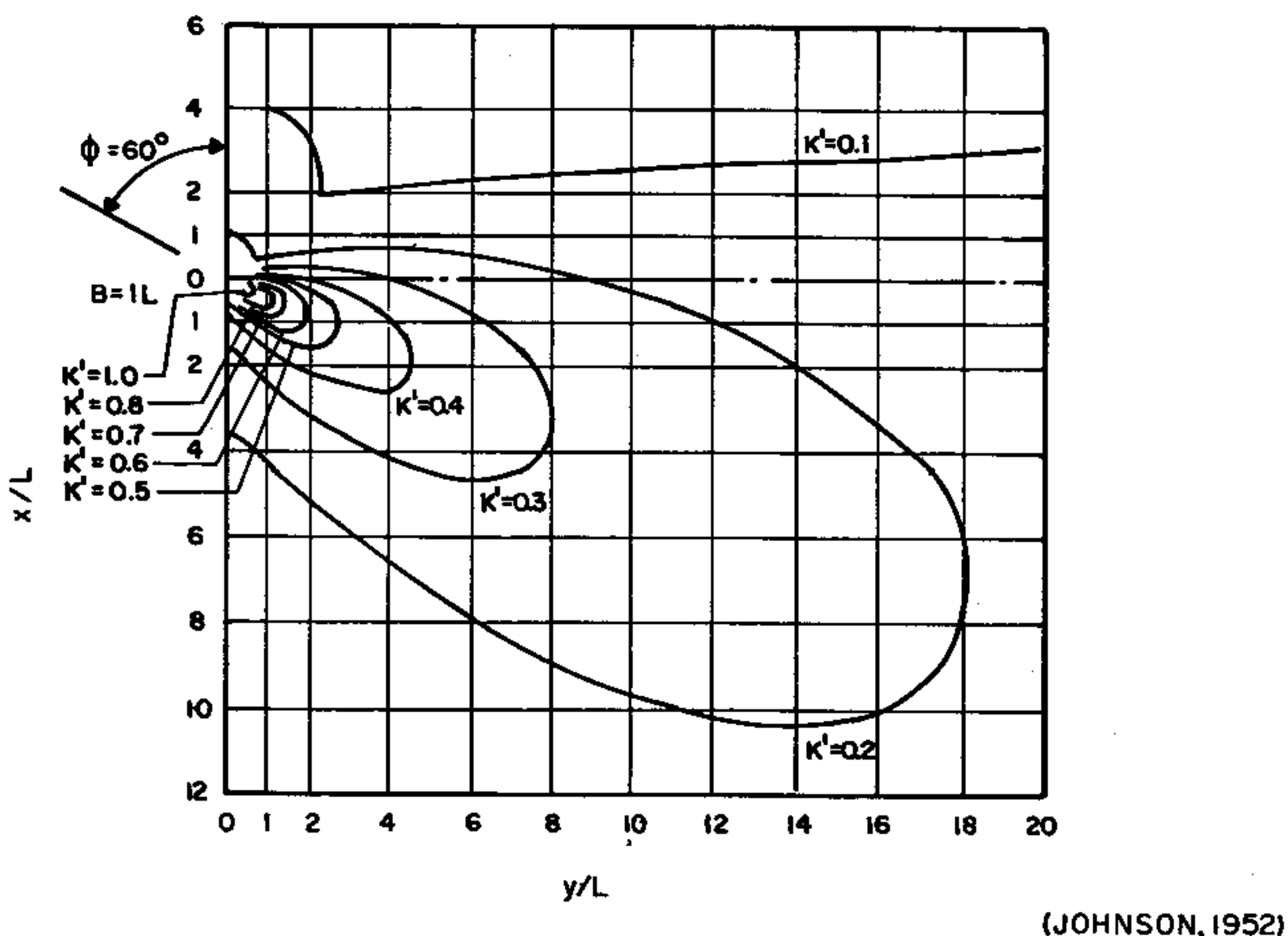
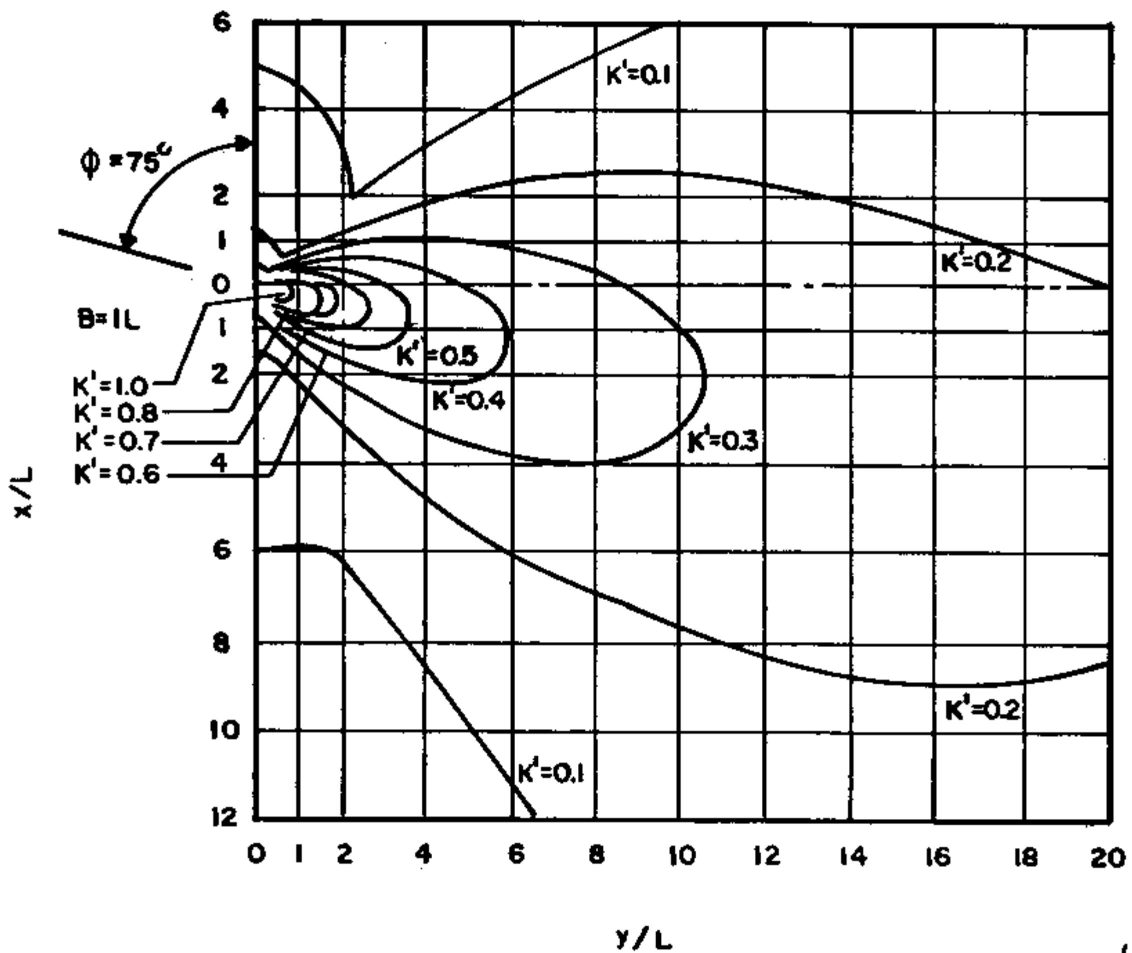


FIGURE 40
Diffraction for a Breakwater Gap of One Wavelength Width and $\phi = 60°$

Width and [phi] = 60 deg.]



(JOHNSON, 1952)

FIGURE 41
Diffraction for a Breakwater Gap of One Wavelength Width and $\phi = 75°$

Width and [phi] = 75 deg.]

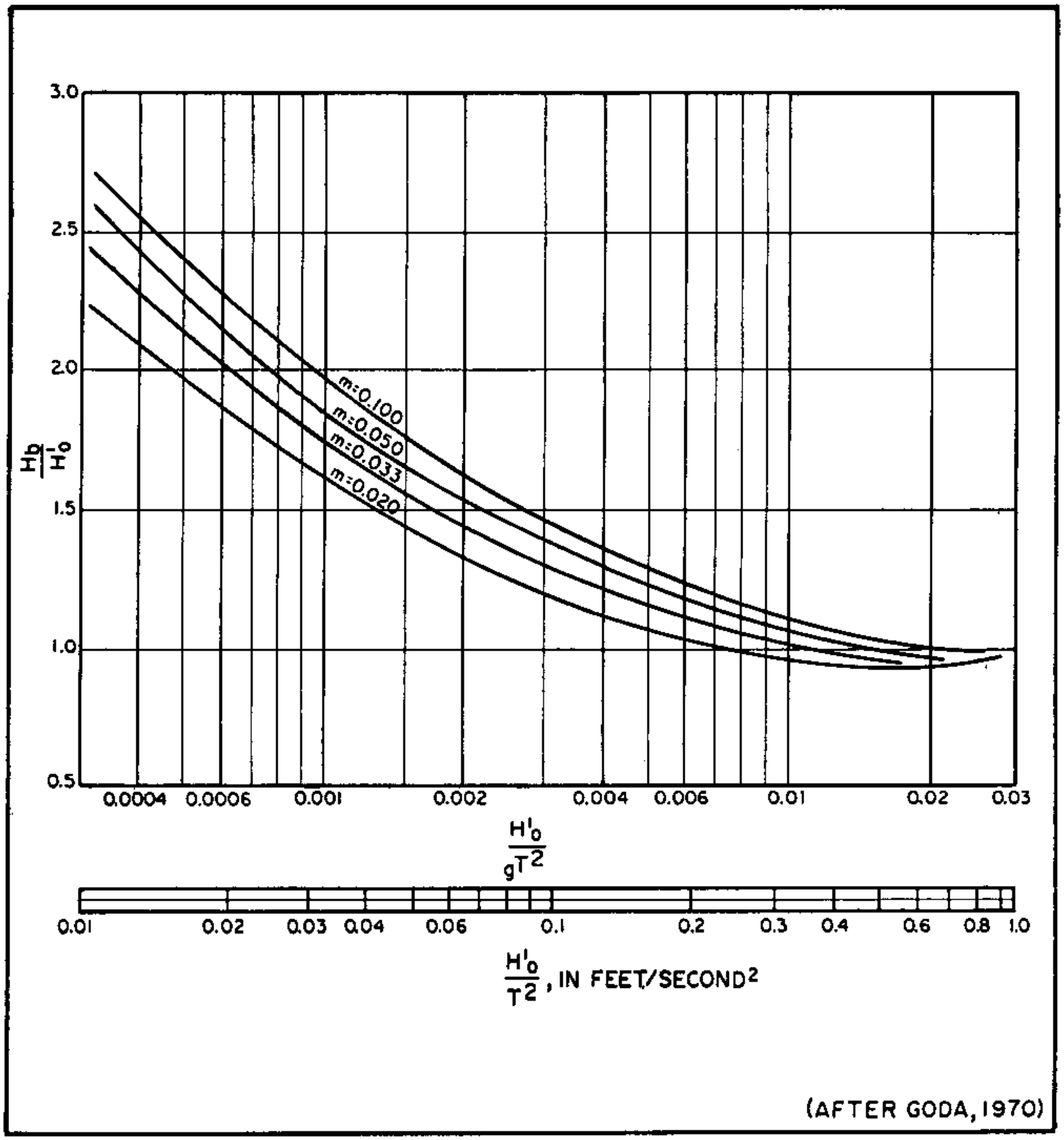


FIGURE 42
Relative Breaker Height, H_b/H'_o, Versus Deepwater Wave Steepness, $H'_o/g\,T^2$

Wave Steepness, H'Úo¿/g TÀ2Ù]

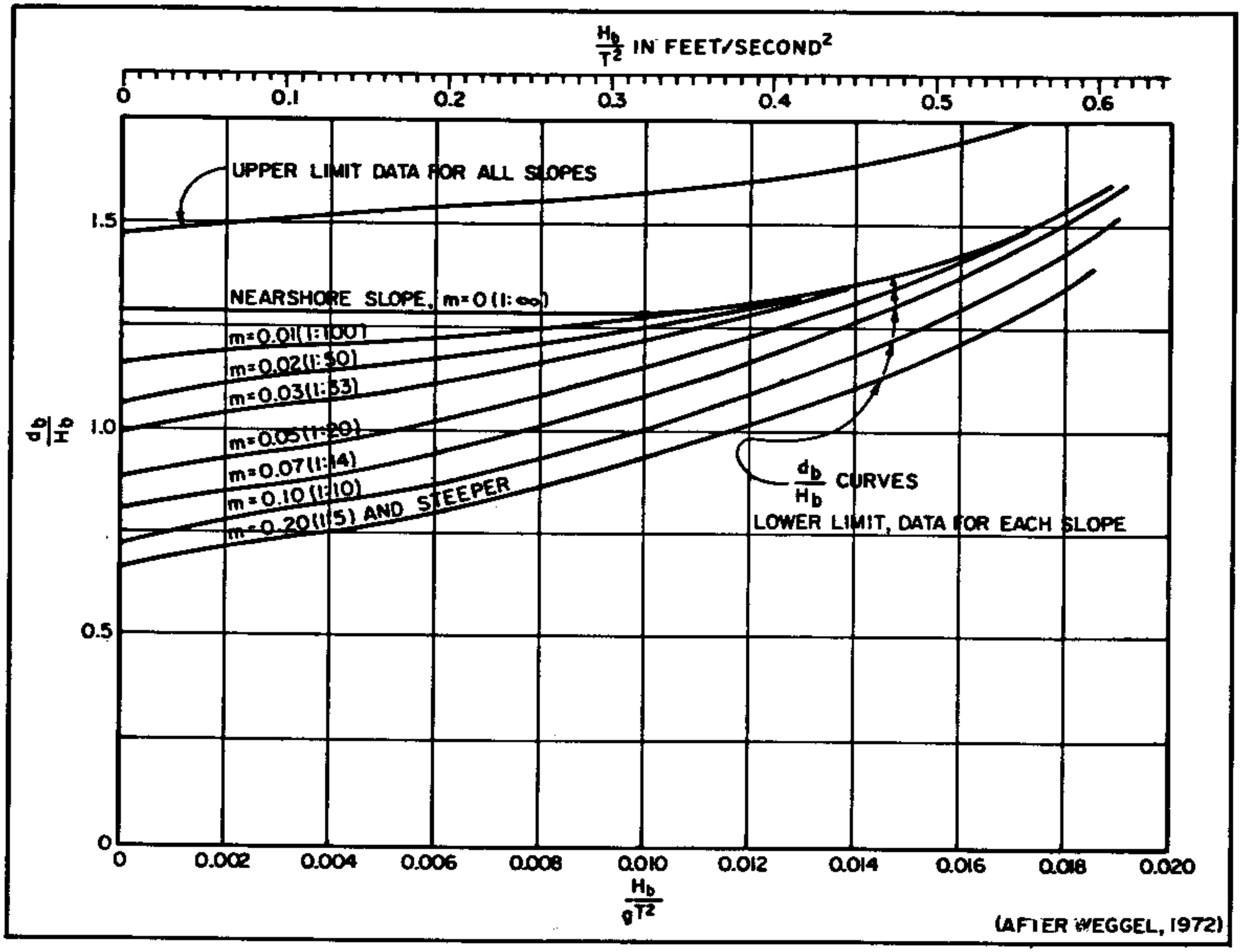


FIGURE 43

d_b/H_b Versus $H_b/g\ T^2$

EXAMPLE PROBLEM 7

Given: a. H'_o = 10 feet
b. T = 10 seconds
c. Bottom slope, m = 0.02

Find: The breaker height, H_b, and breaker depth, d_b.

Solution: (1) Compute $H'_o/g T^2$ = 10/[(32.2)(10)] ^{2} = 0.00311

(2) From Figure 42 for $H'_o/g T^2$ = 0.0031 and m = 0.02:

H_b/H'_o

Then H_b = 1.2 H'_o = (1.2) (10) = 12 feet

(3) Find $H_b/g T^2$:

$H_b/g T^2$ = 12/ [(32. 2) (10)^2] = 0.00373

(4) From Figure 43 for $H_b/g T^2$ = 0.0037 and m = 0.02:

d_b/H_b = 1.13

Then d_b = 1.13 H_b = (1.13) (12) = 13.6 feet

Note: Experimental data were used to develop Figure 43. Breaking waves exhibit a great deal of scatter both in nature and in the model. An upper limit of relative breaker depth, d_b/H_b, is given to indicate at what depth the given wave may start breaking. In this example problem, d_b/H_b (max) = 1.5; therefore, d_b = (1.5) (12) = 18 feet.

(3) Design Wave Height. Waves propagating over a sloping bottom travel a distance of approximately five wave heights (5 H_b) during the breaking process. In general, larger waves can break in he deeper water seaward of a structure. Therefore, a larger wave height seaward of the toe of a structure should be used for the design wave height in limited-water depth situations. For waves approaching over a bottom with constant slope, design wave height should be determined using Figure 44, along with the wave period, T, and depth from SWL at the structure toe, d_s. For waves approaching over an irregularly sloping bottom, either a model study should be conducted, or a representative wave height at a depth five wave heights seaward of the structure should be used.

EXAMPLE PROBLEM 8

Given: A breakwater is located in 10 feet of water (d_s = 10 feet = depth of water at structure toe). The breakwater is fronted by a 1:20 slope (m = 0.05). Analysis of the wave climate

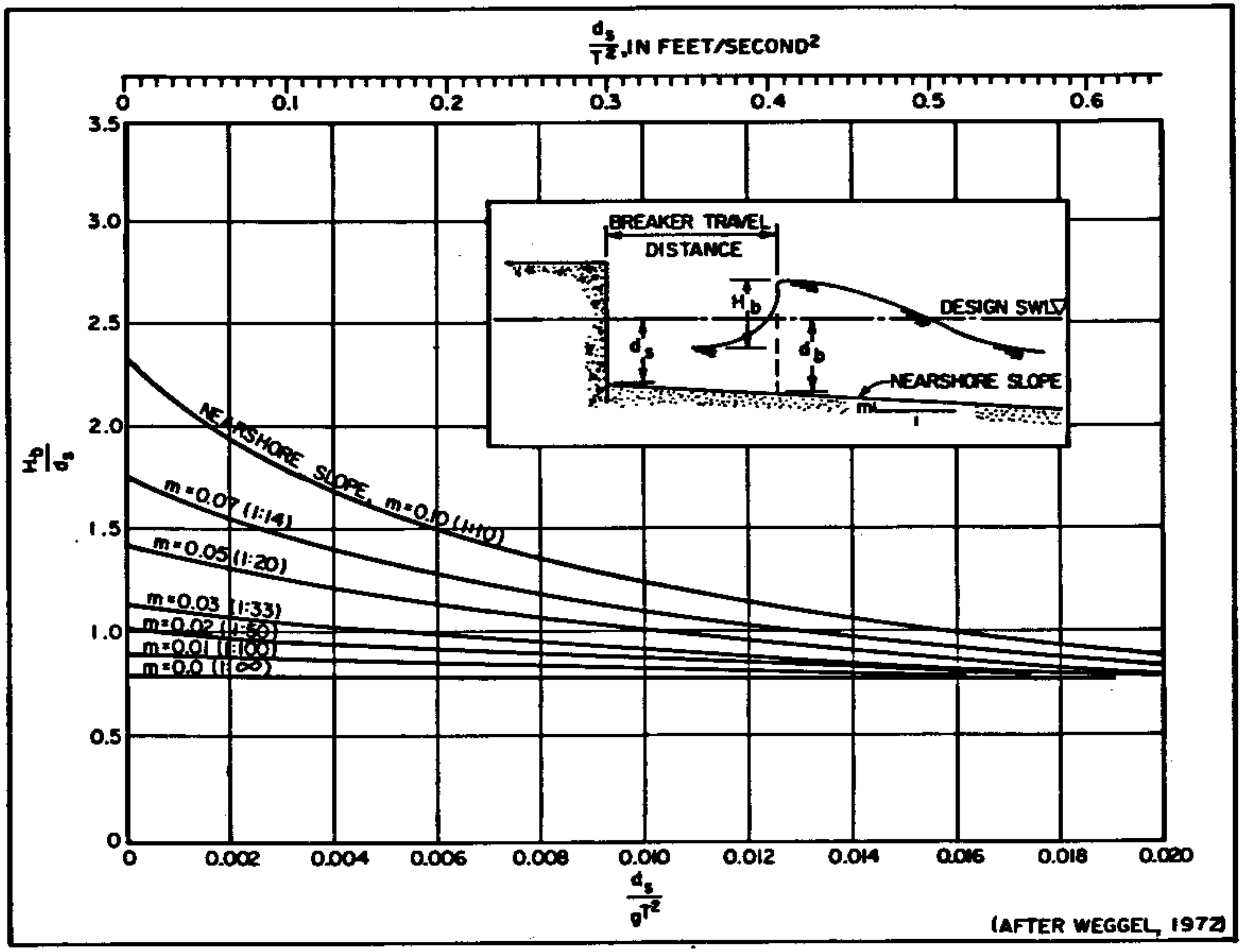


FIGURE 44
H_b/d_s Versus $d_s/g\ T^2$

EXAMPLE PROBLEM 8 (Continued)

indicates that waves approach the site with a period, T, of 5 to 15 seconds.

Find: The maximum breaking-wave height, H_b, at the structure.

Solution: Use T = 5, 10, and 15 seconds

(1) First, calculate H_b for T = 5 seconds

Then $d_s/gT^2 = 10/[(32.2)(5)^2] = 0.0124$

From Figure 44 for $d_s/gT^2 = 0.0124$ and m = 0.05:

$H_b/d_s = 0.95$

Then H_b = deg. 95 d_s = (0.95)(10) = 9.5 feet

(2) Similarly, H_b = 12.5 feet for T = 10 seconds and

H_b = 13.5 feet for T = 15 seconds

THEREFORE: Maximum H_b is 13.5 feet.

(3) The maximum wave that can break on the structure has a height of 13.5 feet. However, if the wave climate is such that a 13.5-foot breaking wave could never occur, then there is no need to design for the maximum H_b. Therefore, it is necessary in this problem to see whether at least a 13.5-foot breaking wave is possible. Check wave-climate data to determine if H'_o with T = 15 seconds can form a 13.5-foot breaking wave. For example, if maximum H'_o = 8 feet, then:

$H'_o/gT^2 = 8/[(32.2)(15)]^2 = 0.00110$

From Figure 42 for $H'_o/gT^2 = 0.0011$ and m = 0.05:

$H_b/H'_o = 1.8$

Then $H_b = 1.8\ H'_o = (1.8)(8) = 14.4$ feet

THEREFORE: A 13.5-foot breaking wave can occur on the structure. (The 14.4-foot wave will break seaward of the structure and therefore is not the design wave.) The design wave is H_b= 13.5 feet.

8. METRIC EQUIVALENCE CHART. The following metric equivalents were developed in accordance with ASTM E-621. These units are listed in the sequence in which they appear in the text of Section 1. Conversions are approximate.

32.2 feet per second^2 = 9.81 meters per second^2

SECTION 2. DESIGN WAVES

1. GENERAL. A coastal structure should be designed to withstand the wave that induces the highest forces on the structure over its economic life. As a general rule of thumb for breakwaters, revetments, and seawalls, the design wave height is the maximum significant wave height that can occur once in about 20 years. Economic considerations involved in selecting the design wave for a given structure must be evaluated in detail. Waves larger than the significant wave will induce some degree of damage to a rubble-mound structure. The cost and extent of repairs to the structure, as well as potential consequences (economic and otherwise) of damage to shore facilities must be evaluated on an individual basis.

Design waves (that is, design wave height and period) can be determined by hindcasting procedures or by analysis of wave observations. When possible, both procedures should be used and the differences between results should be studied to determine which is the more reliable procedure.

2. WAVE DISTRIBUTION.

a. Significant Wave Height. A given sea will contain many waves differing in height, period, and direction of propagation. A spectral approach to design will take some of these variations into account; however, this approach is not commonly used in the United States at present. The deterministic approach does involve some spectral considerations when considering wave-height variations. A representative wave height commonly used in the deterministic approach to design is the significant wave height, H_s. The significant wave height is defined below, followed by a listing of the relationships of other waves in a given sea to the significant height.

(1) $H_s = H_{1/3}$ = average of highest one-third of all waves;
(2) $H = 0.626\ H_s$ = average wave height;
(3) $H_{10} = 1.27\ H_s$ = average of highest 10 percent of all waves; and
(4) $H_1 = 1.67\ H_s$ = average of highest 1 percent of all waves.

The wave height reported by observers on ships has been shown to approximate the significant wave height.

b. Variations in Period or Direction. The wave period is normally taken as a subjective period associated with a hindcasting procedure. Wave direction can vary as much as 90 deg. on either side of the principal wind direction. Variations in period or direction within a given sea condition are generally not taken into consideration in calculations. However, they should be considered if the design is critical to minor variations in either one.

3. WAVE HINDCASTING.

a. Hindcast Parameters. Wave hindcasting is the calculation of wave characteristics that probably occurred in the past based on synoptic wind data. Wave hindcasting is an art that is continuing to evolve through theoretical considerations coupled with observations. The important parameters required to estimate a wave condition for a given storm or wind condition are listed below.

(1) Fetch. Fetch, or fetch length, is the area or distance over which a wind field generates seas. The greater the distance, the larger the waves and the longer the period will be for given windspeed and wind duration. The growth of waves (that is, wave heights becoming larger and wave periods becoming longer) may be limited by fetch length.

(2) Windspeed. Windspeed is the sustained windspeed at 32.8 feet, or 10 meters, above the sea surface.

(3) Direction of Wind. Waves are assumed to propagate with the direction of the wind. However, seas may propagate up to 45 deg. from the principal wind direction.

(4) Wind Duration. Waves increase in height and period for given windspeed and fetch, until they become fully arisen. Thereafter, a further increase in duration does not increase wave height or period.

(5) Water Depth. Bottom friction and percolation retard the growth of waves in shallow water.

(6) Decay Distance. Once the waves leave the generating area they decrease in height and the period increases.

b. Hindcasting Procedure. Hindcasts may be made by inferring wind fields from synoptic weather charts or by transforming windspeed data from wind gages. Synoptic weather charts may be obtained from the U.S. Navy Fleet Numerical Weather Central (FNWC) for a given storm. Usually, several years of storms must be analyzed. Description of the procedure for use of synoptic weather charts lies beyond the scope of this manual. (More detailed hindcasting procedures can be found in the Shore Protection Manual (1977).) Where the fetch is defined by an enclosed body of water, such as a bay or lake, and wind observations are available, the procedures outlined below should be employed.

(1) Windspeed. In order to determine the appropriate windspeed for use in hindcasting procedures, depending on the type of wind record available, the following steps should be taken. The result will be the final adjusted windspeed, U_A.

(a) Correction for elevation. If the wind is recorded at an elevation other than 10 meters, then the windspeed at 10 meters, U_{10}, is determined using the following equation:

$$U_{10} = \left(\frac{10}{z}\right)^{1/7} U_z \qquad (2\text{-}1)$$

WHERE: U_{10} = windspeed at elevation of 10 meters

z = elevation of recorded wind, in meters (z must be < 100 meters for this method to be valid (Bretschneider, 1969))

U_z = windspeed at elevation z

(b) Correction for duration. Recorded windspeeds may vary in definition. For example, recorded windspeeds may be fastest mile, 5-minute average, or instantaneous maximum gust. For use in hindcasting, windspeed must be adjusted so that the average time is equal to or greater than the minimum duration required for the wind to fully develop the waves. This involves an iterative procedure which will be discussed in Example Problem 9. Figure 45 is used to adjust the recorded windspeed of given duration (adjusted to 10-meter elevation) to the value of windspeed, U_t, at the desired duration, where t = wind duration.

$$U_{(t_{desired})} = \left[\frac{C_{(t_{desired})}}{C_{(t_{given})}} \right] \left[U_{(t_{given})} \right] \qquad (2\text{-}2)$$

WHERE: $U_{(t_{desired})}$ = windspeed at desired duration, adjusted for elevation and duration

t = wind duration

$C_{(t_{desired})}$ = conversion factor (found from Figure 45)

$C_{(t_{given})}$ = conversion factor (found from Figure 45)

$U_{(t_{given})}$ = windspeed at given duration, adjusted for elevation

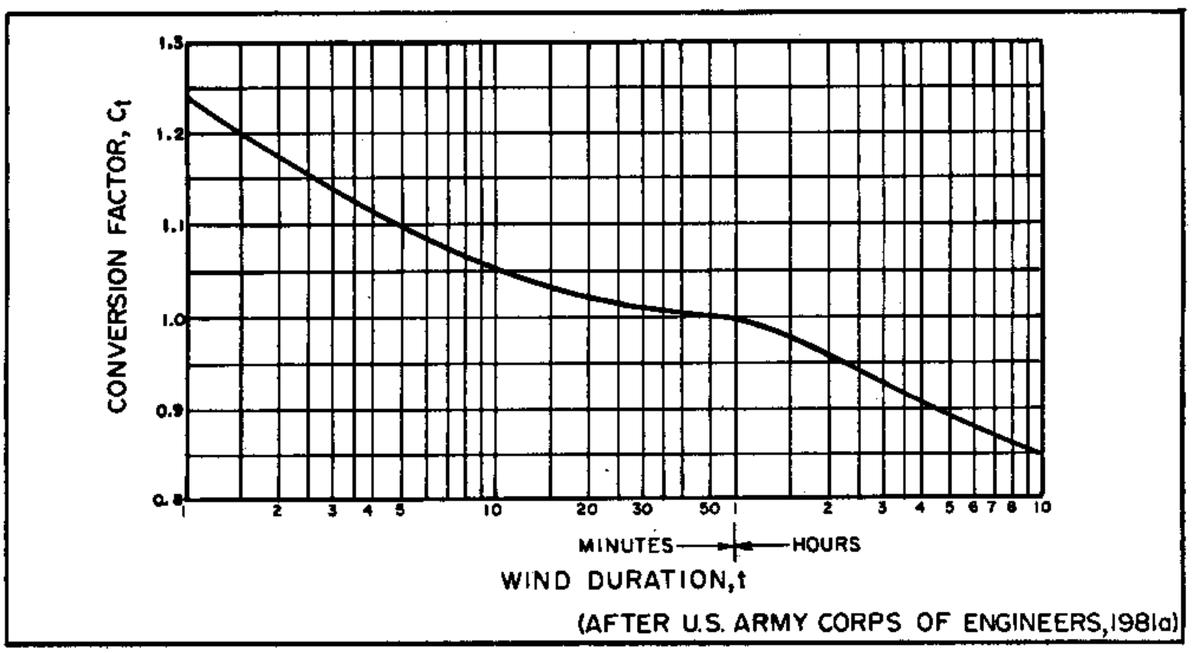


FIGURE 45
Windspeed Conversion Factor, C_t, as a Function of Wind Duration, t

Duration, t]

(c) Correction for overland-overwater effects. Windspeed recorded overland (U_L) must be adjusted to obtain the overwater windspeed U_W. This can be achieved using the following procedure.

If the fetch length is less than or equal to 10 miles:

$$U_W = 1.1\ U_L \text{ (for fetch length < / = 10 miles)} \qquad (2\text{-}3)$$

WHERE: U_W = overwater windspeed

U_L = overland windspeed adjusted for elevation and duration

If the fetch length is greater than 10 miles:

$$U_W = R\ U_L \text{ (for fetch length > 10 miles)} \qquad (2\text{-}4)$$

WHERE: $R = U_W/U_L$ = ratio of overwater windspeed to overland windspeed (found from Figure 46)

(d) Correction for nonconstant drag coefficient. Winds must be adjusted for nonconstant coefficient of drag. This can be accomplished using the following equations:

$$U'_A = 0.608\ U_W^{1.23} \text{ (in knots)} \qquad (2\text{-}5)$$

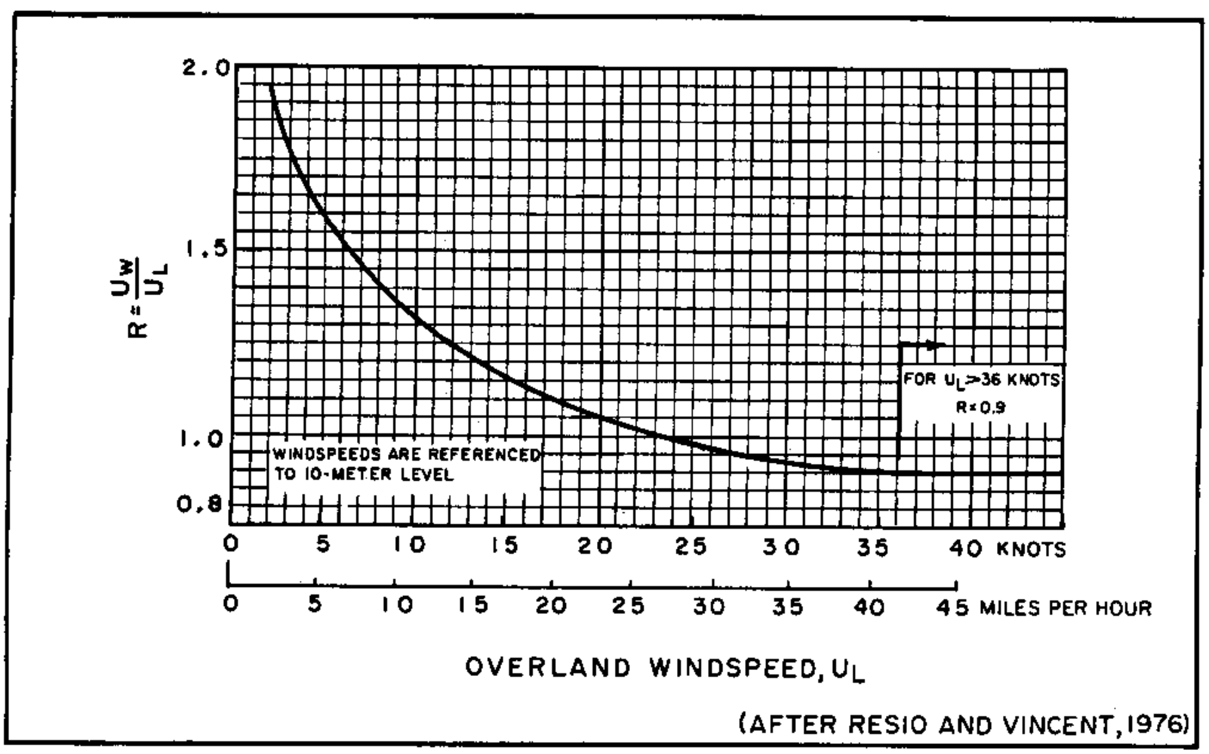


FIGURE 46
Ratio, R, of Overwater, U_W, to Overland, U_L, Windspeed as a Function of Overland Windspeed, U_L

Windspeed as a Function of Overland Windspeed, UÚL¿]

or

$$U'_A = 0.589\ U_W^{1.23} \text{ (in statute miles per hour)} \qquad (2\text{-}6)$$

WHERE: U_A = windspeed corrected for nonconstant drag coefficient

U_W = windspeed adjusted for elevation, duration, and overland overwater effects

If the fetch length is less than or equal to 10 miles, then no further adjustment is necessary, and the final adjusted windspeed, U_A, is:

$U_A = U'_A$ (for fetch length < / = 10 miles)

WHERE: U_A = final adjusted windspeed used for hindcasting (for fetch length < / = 10 miles)

(e) Correction for air-sea temperature difference. For fetch lengths greater than 10 miles, an adjustment resulting from the air-sea temperature difference must be made to the windspeed. If the temperature difference is specifically known in degrees centigrade, the amplification ratio, R_T, is determined from Figure 47 from known values of air temperature, T_a, minus water temperature, T_s. The resulting windspeed is:

$$U_A = R_T\ U'_A \text{ (for fetch length > 10 miles)} \qquad (2\text{-}7)$$

WHERE: U_A = final adjusted windspeed used for hindcasting (for fetch length > 10 miles)

R_T = amplification ratio (found from Figure 47)

U_A = windspeed adjusted for elevation, duration, overland-overwater effects, and nonconstant drag coefficient

If the air-sea temperature difference is not known, the value of R_T may be determined by estimating the condition of the atmospheric boundary layer. If the air is warmer than the water, then the atmospheric boundary layer is assumed stable, and U_A is determined as follows:

$$U_A = 0.9\ U'_A \text{ for stable atmospheric boundary layer} \qquad (2\text{-}8)$$

If the air and water are at the same temperature, then the atmospheric boundary layer is assumed to have neutral stability. U_A for a neutrally stable atmospheric boundary layer is determined as follows:

$$U_A = 1.0\ U'_A \text{ for neutrally stable atmospheric boundary layer} \qquad (2\text{-}9)$$

If the air is cooler than the water, then the atmospheric boundary layer is assumed unstable. For an unstable atmospheric boundary layer:

$$U_A = 1.1\ U'_A \text{ for unstable atmospheric boundary layer} \qquad (2\text{-}10)$$

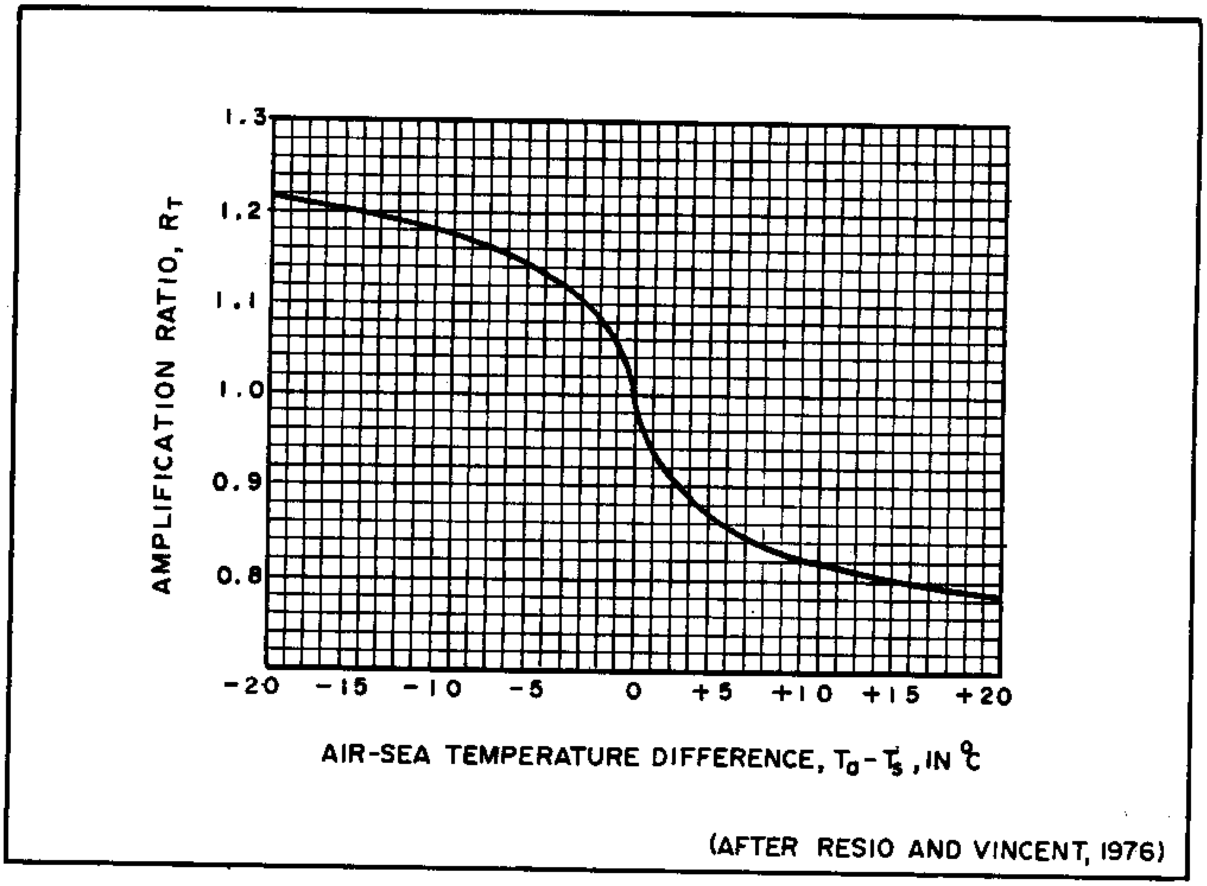


FIGURE 47
Amplification Ratio, R_T, as a Function of Air-Sea Temperature Difference, $T_a - T_s$

Temperature Difference, TÚa¿ - TÚs¿]

(2) Fetch Length. Determine appropriate fetch length, F, by taking the straight-line distance along the axis of the wind to the opposite shore or boundary.

(3) Use of Hindcasting Charts. Values of the significant wave height, HÚs¿, and the wave period, TÚp¿ associated with the highest peak of the wave spectrum, can be determined for a given adjusted windspeed, UÚA¿, fetch length, F, water depth, d, and minimum wind duration, t, using the hindcasting charts presented in Figures 48-58. For water depths greater than 50 feet, wave generation is not greatly affected by depth variations and Figure 48 is used. For water depths less than or equal to 50 feet, Figures 49 through 58 are used. These hindcasting charts plot fetch length, F, on the abscissa and adjusted windspeed, UÚA¿, on the ordinate. Also plotted on the figures are isolines of significant wave height, HÚs¿, peak spectral period, TÚP¿, and minimum duration, t.

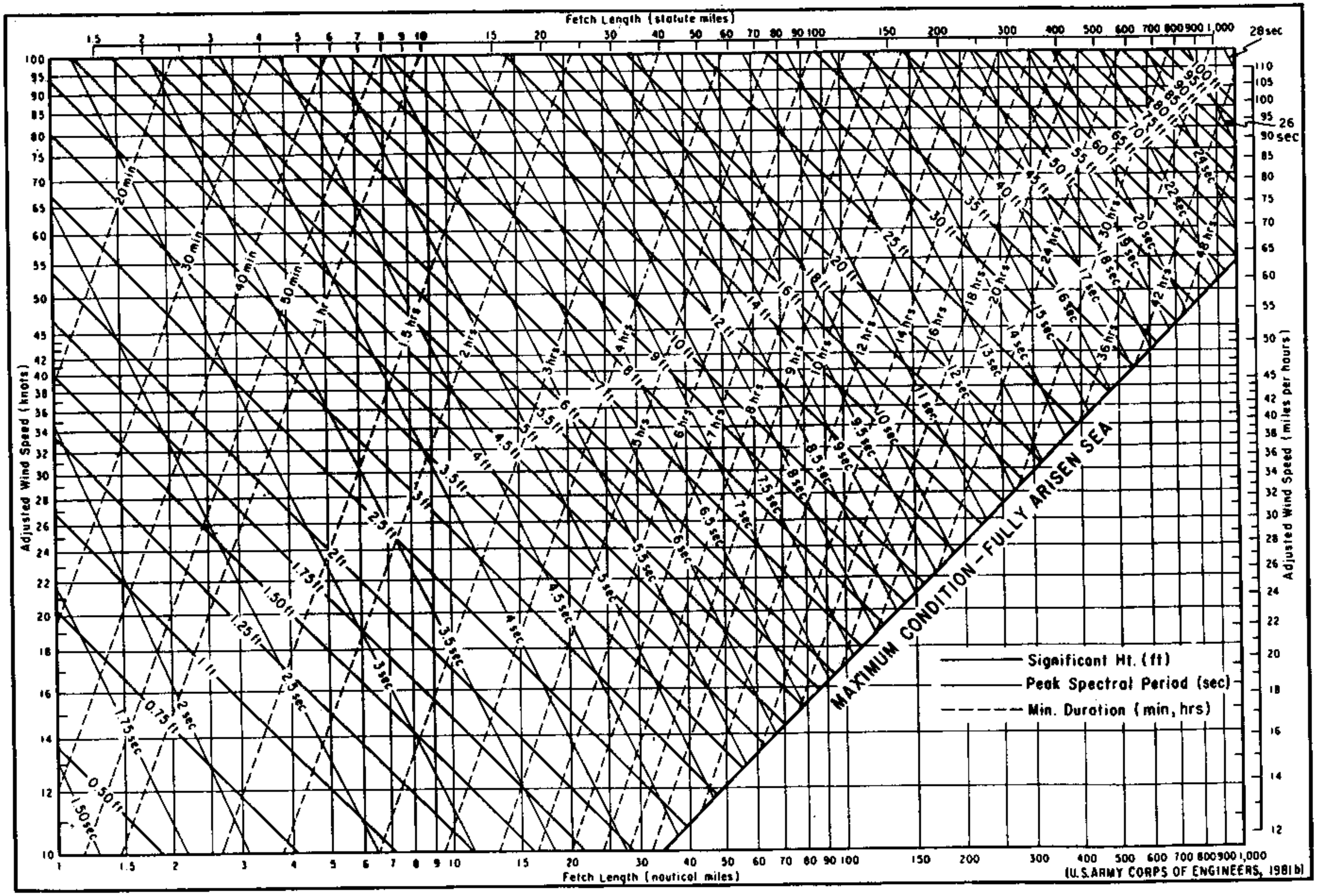
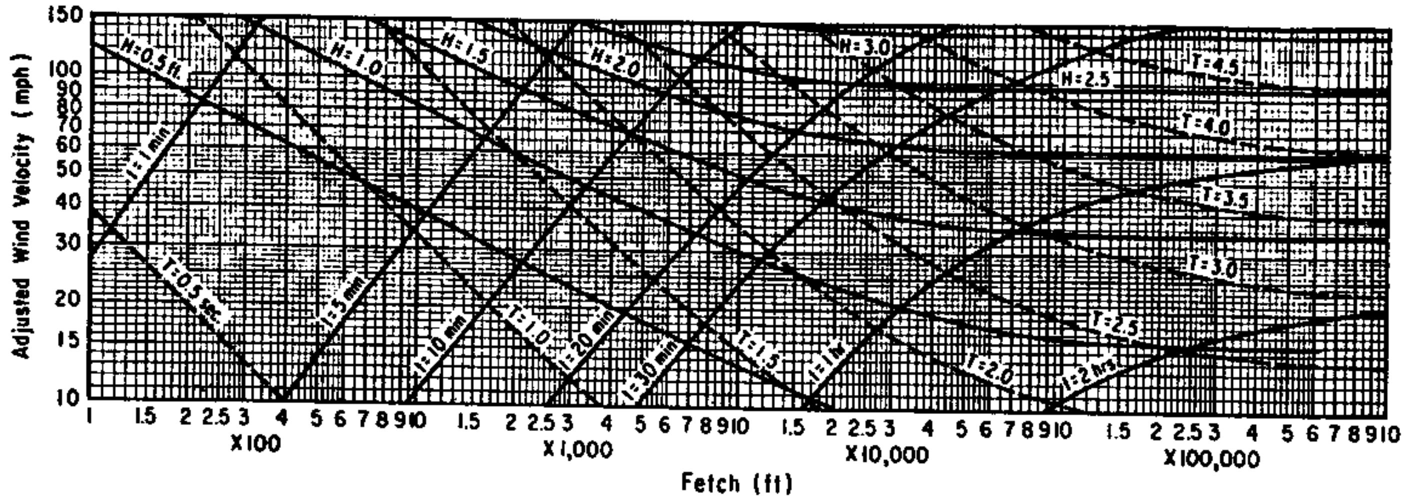


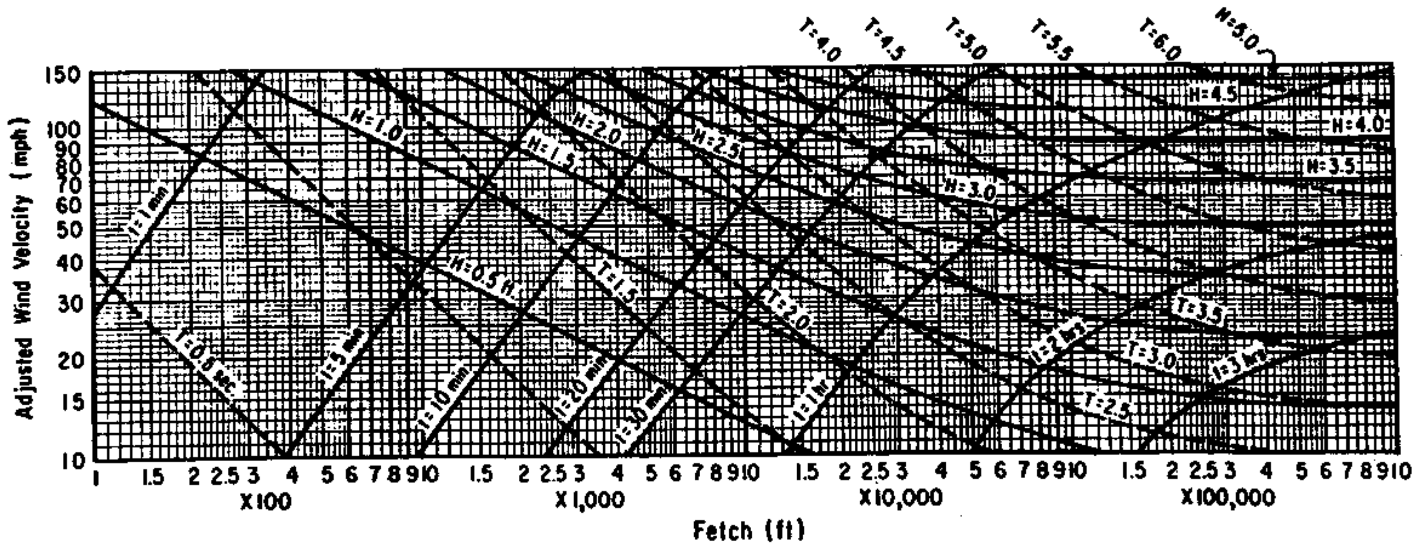
FIGURE 48
Hindcasting Chart for Deepwater Waves (Water Depth > 50 Feet)



NOTE: WAVES IN A WATER DEPTH OF 5 FEET WITH WAVE PERIODS LESS THAN 1.4 SECONDS ARE CONSIDERED TO BE DEEPWATER WAVES, I.E. $d/T^2 > 2.56$

(U.S. ARMY CORPS OF ENGINEERS, 1981c)

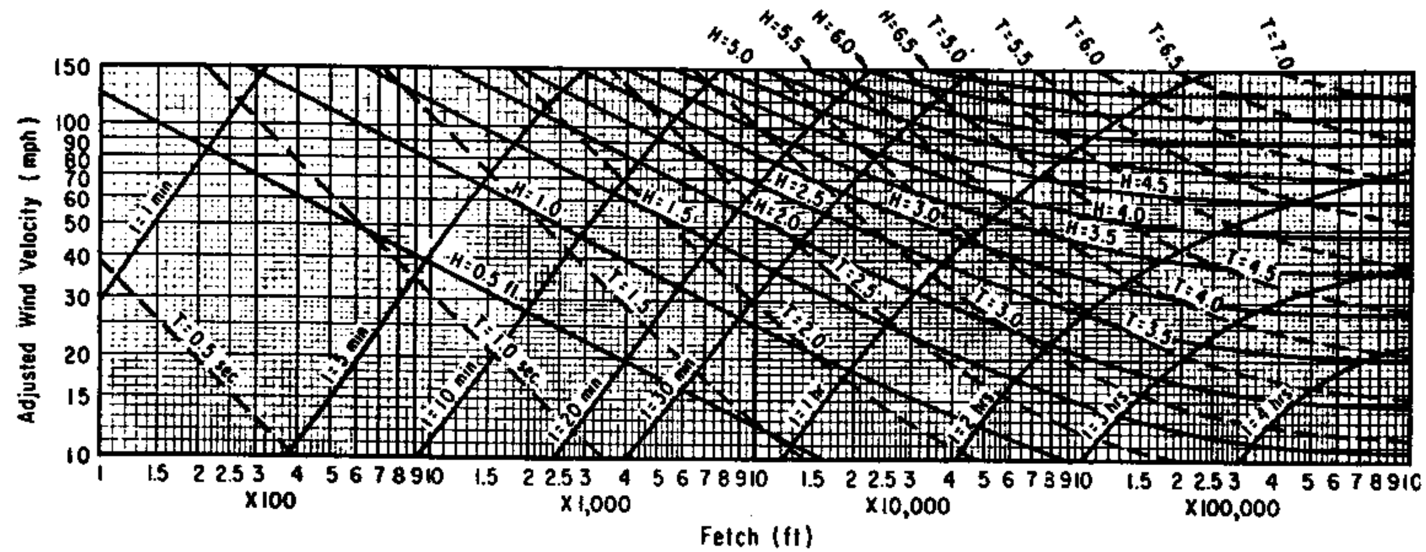
FIGURE 49
Hindcasting Chart for Shallow-Water Waves; Constant Depth = 5 Feet



NOTE: WAVES IN A WATER DEPTH OF 10 FEET WITH WAVE PERIODS LESS THAN 2.0 SECONDS ARE CONSIDERED TO BE DEEPWATER WAVES, I.E. $d/T^2 > 2.56$

(U.S. ARMY CORPS OF ENGINEERS, 1981c)

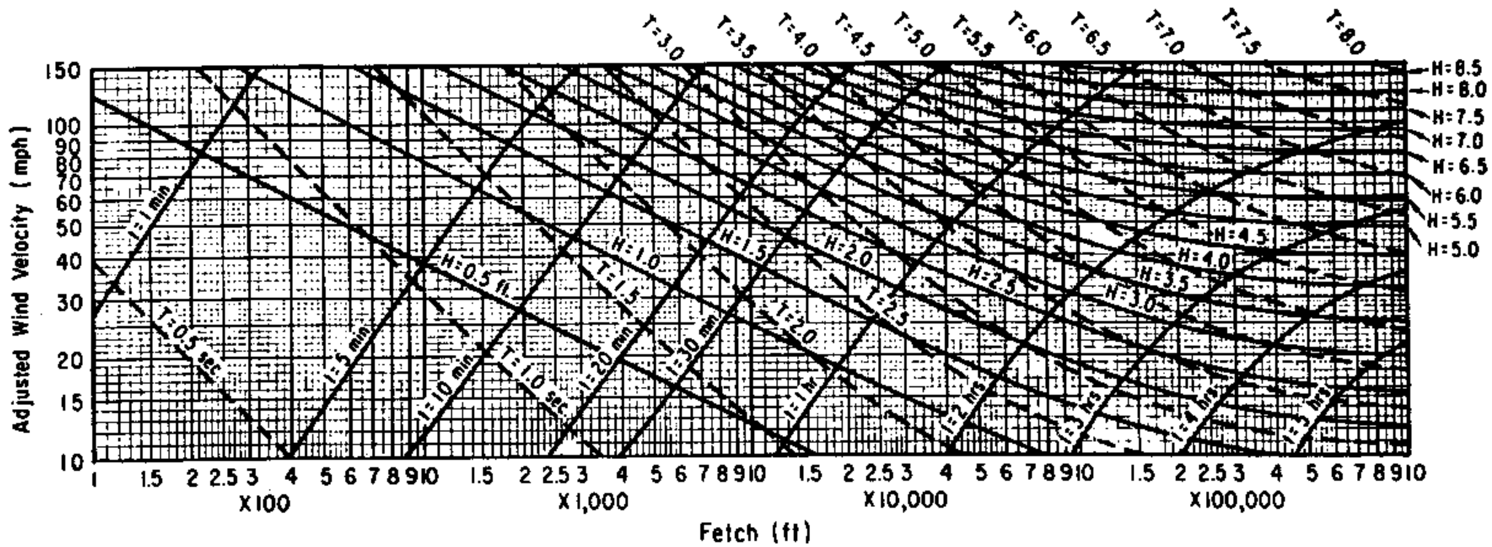
FIGURE 50
Hindcasting Chart for Shallow-Water Waves; Constant Depth = 10 Feet



NOTE: WAVES IN A WATER DEPTH OF 15 FEET WITH WAVE PERIODS LESS THAN 2.4 SECONDS ARE CONSIDERED TO BE DEEPWATER WAVES, I.E. $d/T^2 > 2.56$

(U. S. ARMY CORPS OF ENGINEERS, 1981c)

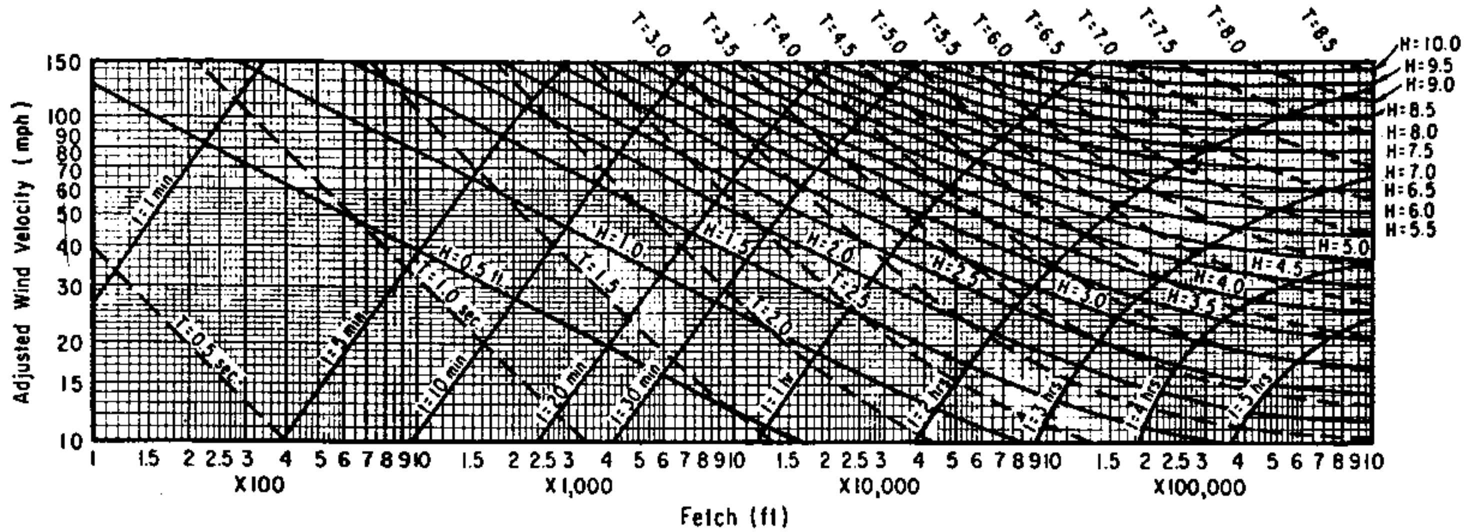
FIGURE 51
Hindcasting Chart for Shallow-Water Waves; Constant Depth = 15 Feet



NOTE: WAVES IN A WATER DEPTH OF 20 FEET WITH WAVE PERIODS LESS THAN 2.8 SECONDS ARE CONSIDERED TO BE DEEPWATER WAVES, I.E. $d/T^2 > 2.56$

(U.S. ARMY CORPS OF ENGINEERS, 1981c)

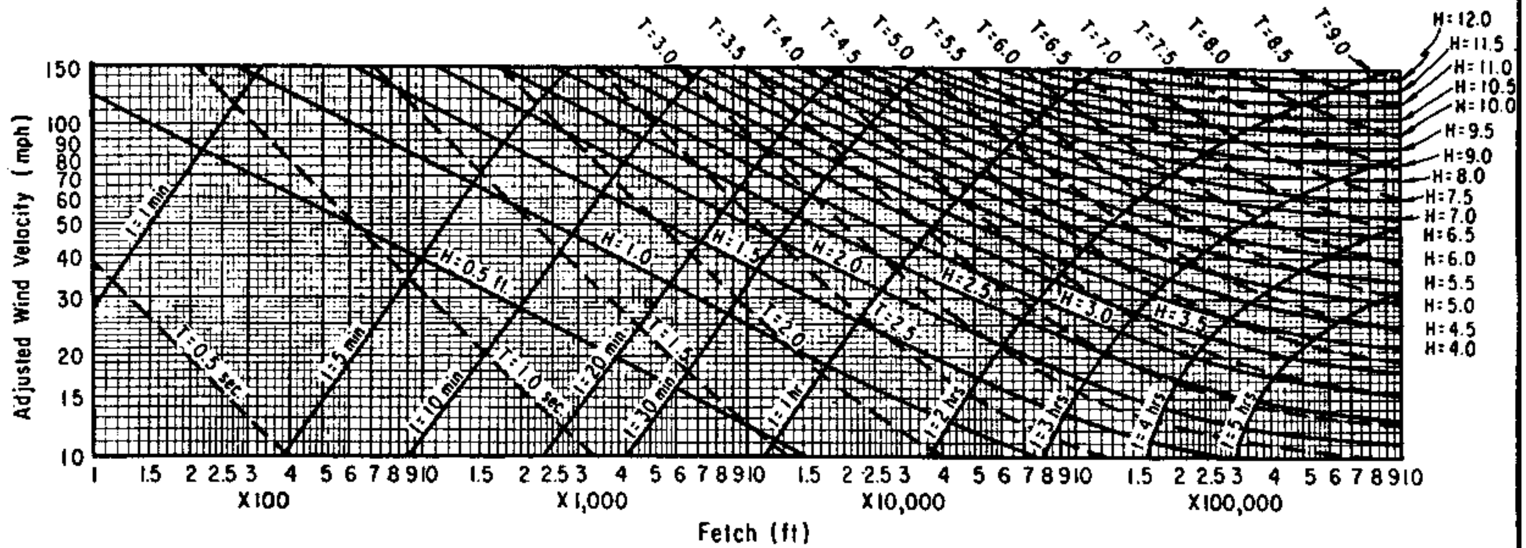
FIGURE 52
Hindcasting Chart for Shallow-Water Waves; Constant Depth = 20 Feet



NOTE: WAVES IN A WATER DEPTH OF 25 FEET WITH WAVE PERIODS LESS THAN 3.1 SECONDS ARE CONSIDERED TO BE DEEPWATER WAVES, I.E. $d/T^2 > 2.56$

(U. S. ARMY CORPS OF ENGINEERS, 1981c)

FIGURE 53
Hindcasting Chart for Shallow-Water Waves; Constant Depth = 25 Feet

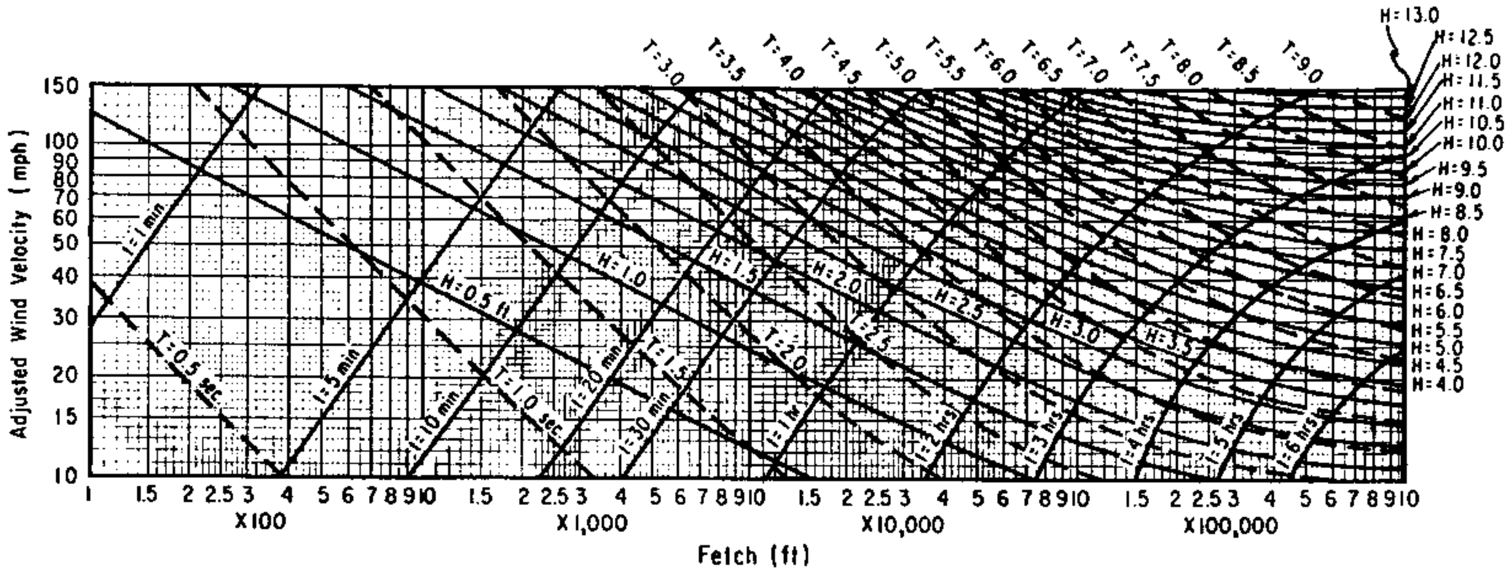


NOTE: WAVES IN A WATER DEPTH OF 30 FEET WITH WAVE PERIODS LESS THAN 3.4 SECONDS ARE CONSIDERED TO BE DEEPWATER WAVES, I.E. $d/T^2 > 2.56$

(U.S. ARMY CORPS OF ENGINEERS, 1981c)

FIGURE 54

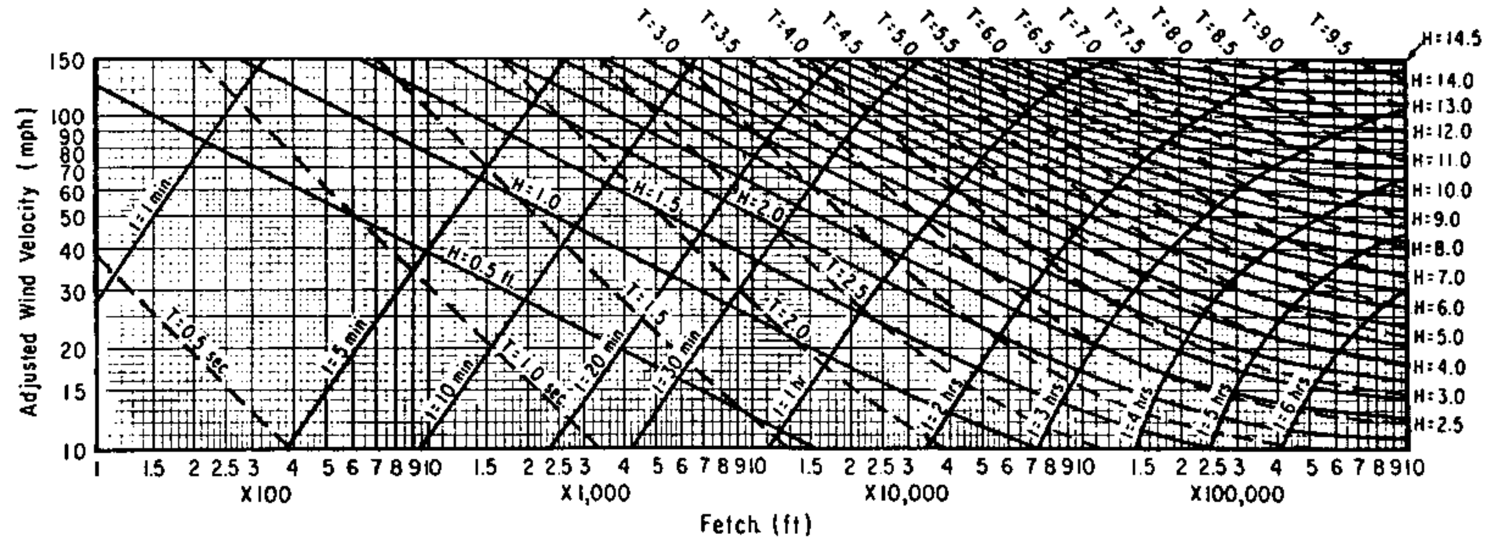
Hindcasting Chart for Shallow-Water Waves; Constant Depth = 30 Feet



NOTE: WAVES IN A WATER DEPTH OF 35 FEET WITH WAVE PERIODS LESS THAN 3.7 SECONDS ARE CONSIDERED TO BE DEEPWATER WAVES, I.E. $d/T^2 > 2.56$

(U.S. ARMY CORPS OF ENGINEERS, 1981c)

FIGURE 55
Hindcasting Chart for Shallow-Water Waves; Constant Depth = 35 Feet

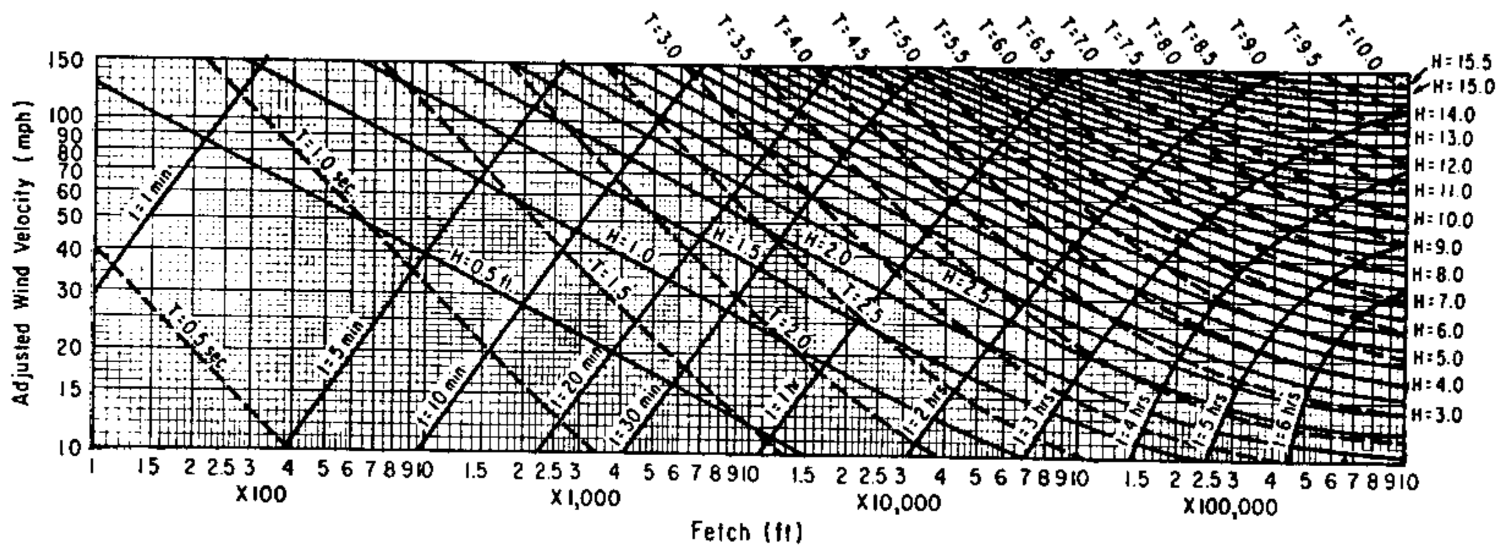


NOTE: WAVES IN A WATER DEPTH OF 40 FEET WITH WAVE PERIODS LESS THAN 4.0 SECONDS ARE CONSIDERED TO BE DEEPWATER WAVES, I.E. $d/T^2 > 2.56$

(U.S. ARMY CORPS OF ENGINEERS, 1981c)

FIGURE 56

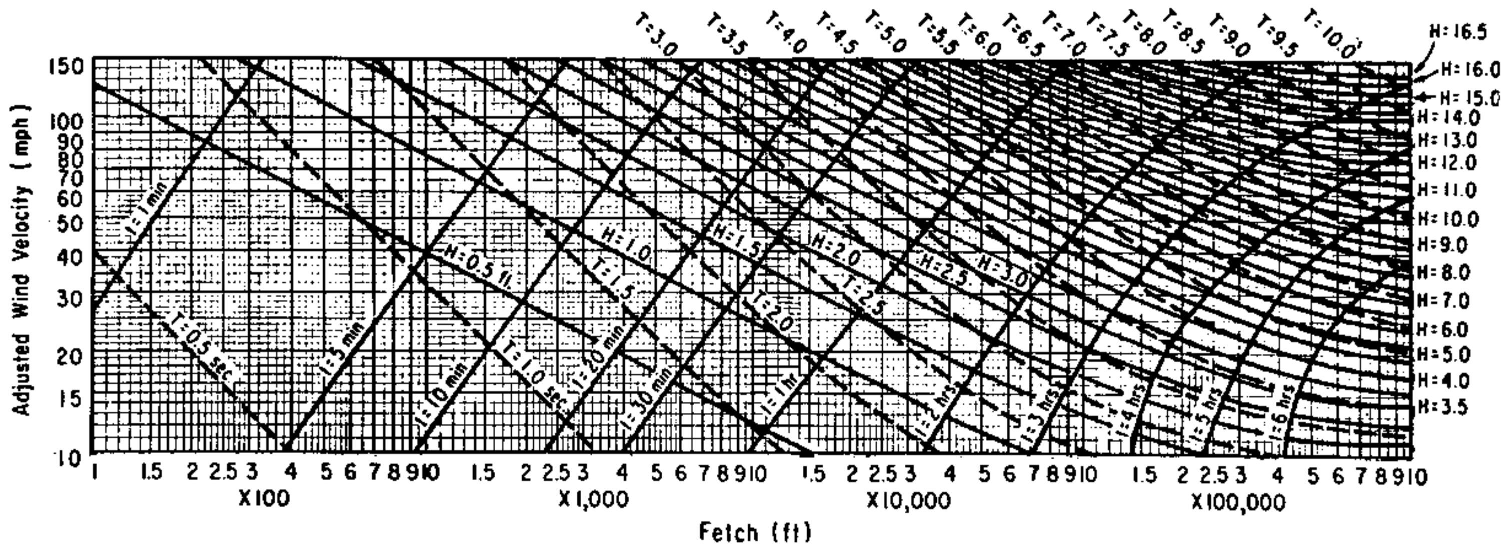
Hindcasting Chart for Shallow-Water Waves; Constant Depth = 40 Feet



NOTE: WAVES IN A WATER DEPTH OF 45 FEET WITH WAVE PERIODS LESS THAN 4.2 SECONDS ARE CONSIDERED TO BE DEEPWATER WAVES, I.E. $d/T^2 > 2.56$

(U.S. ARMY CORPS OF ENGINEERS, 1981c)

FIGURE 57
Hindcasting Chart for Shallow-Water Waves; Constant Depth = 45 Feet



NOTE: WAVES IN A WATER DEPTH OF 50 FEET WITH WAVE PERIODS LESS THAN 4.4 SECONDS ARE CONSIDERED TO BE DEEPWATER WAVES, I.E. $d/T^2 > 2.56$

(U.S. ARMY CORPS OF ENGINEERS, 1981c)

FIGURE 58
Hindcasting Chart for Shallow-Water Waves; Constant Depth = 50 Feet

Wave conditions (or seas) can be either fetch-limited or duration-limited. For hindcasting fetch-limited seas, one enters the abscissa with the fetch length and the ordinate with the adjusted windspeed. Where these intersect, the values for H_s, T_p, and t are read from the chart. When hindcasting from wind observations recorded at an arbitrary duration, the winds should be adjusted to the duration, t. If hindcasting is carried out for a particular storm where winds are known to blow for a specific duration, the duration of the wind must equal or exceed the minimum duration, t, in order for the waves to reach H_s and T_p for a given F and U_A. If the duration of the wind is less than t, then the seas are termed duration-limited. To obtain H_s and T_p for duration-limited seas, one enters the hindcasting chart ordinate with U_A and proceeds to the intersection of the duration value equal to the duration of the storm.

Bathymetry may vary considerably over a large fetch. When the bathymetry contains extended regions of depths less than or equal to 50 feet, the average depth, at the design water level (see DM-26.1, Section 2.7., for a discussion of design water level) may be used. A better approximation for use in hindcasting waves in variable depths (< / = 50 feet) is to divide the fetch into discrete intervals of constant depth. Then starting with the first fetch interval, F_1, hindcast the significant wave height, H_s, for the given U_A, F_1, d_1, and t. Using the hindcasted value of H_s at the end of the first interval and the adjusted windspeed, U_A, enter the hindcasting chart for the depth of the next interval, d_2, and determine the corresponding fetch length, F'_1. This is the fetch length required to generate H_s if the water depth had been d_2 in the first interval. To this fetch length, F'_1, add the fetch length, F_2, for the second interval; use the resulting value along with U_A and d_2 for hindcasting H_s at the end of the second interval. Repeat this process for the all the fetch intervals making up the total fetch length. The peak spectral period, T_p, and the minimum duration, t, are assumed to be the values obtained at the end of the last fetch interval. The hindcasting procedures are illustrated in Example Problems 9 and 10.

c. Other Considerations. The preceding procedures give estimates of wave characteristics at the end of the fetch. Refraction, shoaling, diffraction, wave-breaking, and economical analyses must be performed to determine the design wave. As the wave propagates out of the generating area, it decays. In general design calculations, wave decay is not important; however, if the wave leaves the generating area and travels great distances over shallow water, wave decay should be considered. The Shore Protection Manual (1977) gives specific guidance on this topic. Waves from tropical cyclones, known as hurricanes or typhoons, may be calculated by equations given in the Shore Protection Manual.

EXAMPLE PROBLEM 9

Given:
a. Hourly average windspeed, U_z = 45 knots, measured over land at 10 feet above the ground (t = 1 hour; z = 10 feet)
b. Water temperature, T_s = 15 deg. Centigrade and air temperature, T_a = 24.5 deg. Centigrade
c. Fetch length, F = 35 nautical miles
d. Average water depth, d = 75 feet

EXAMPLE PROBLEM 9 (Continued)

Find: The adjusted windspeed, U_A, the significant wave height, H_s, and peak spectral period, T_p.

Solution: (1) Correct for elevation, using Equation (2-1):

$$U_{10} = (10/z)^{1/7} U_z$$

$$z = (10 \text{ feet}) (0.3048 \text{ meters/foot}) = 3.048 \text{ meters}$$

$$U_{10} = (10/3.048)^{1/7}(45) = 53.3 \text{ knots}$$

(2) Correct for duration, using Equation (2-2):

The proper duration to use is the minimum duration, t, found from the hindcasting chart for the given conditions. At this point a duration must be assumed. After hindcasting, the minimum duration, t, read from the chart should be equal to the assumed duration. If not, the process should be reiterated until the values of duration are equal.

From Figure 48 for U_{10} = 53.3 knots and F = 35 nautical miles:

$t = 4.4$ hours = desired duration = $t_{desired}$

From Figure 45 for t = 4. 4 hours (desired duration):

Conversion factor, $C_t = C_{(t_{desired})} = 0.9$,
where t = 4.4 hours;

therefore $C_{(t = 4.4 \text{ hours})} = 0.9$

From Figure 45 for t = 1 hour (given duration):

Conversion factor, $C_t = C_{(t_{given})} = 1.0$,
where t = 1 hour;

therefore, $C_{(t = 1 \text{ hour})} = 1.0$

$$U_{(t = 4.4 \text{ hours})} = \left[\frac{C_{(t = 4.4 \text{ hours})}}{C_{(t = 1 \text{ hour})}} \right] \left[U_{(t = 1 \text{ hour})}\right]$$

$$U_t = \left(\frac{0.9}{1.0} \right) (53.3) = 47.97 \text{ knots}$$

(3) Correct for overland-overwater effects:

From Figure 46 for $U_t = U_L = 47.97$ knots:

R = 0.9

EXAMPLE PROBLEM 9 (Continued)

Using Equation (2-4):

$U_W = R\ U_L$

$U_W = (0.9)\ (47.97) = 43.17$ knots

(4) Correct for nonconstant drag coefficient:

Using Equation (2-5):

$U'_A = 0.608\ U_W^{1.23}$

$U'_A = (0.608)\ (43.17)^{1.23} = 62.40$ knots

(5) Correct for air-sea temperature difference:

$T_a = 24.5$ deg. C

$T_s = 15$ deg. C

$T_a - T_s = 24.5$ deg. C $- 15$ deg. C $= + 9.5$ deg. C

From Figure 47 for $T_a - T_s = + 9.5$ deg. C:

$R_T = 0.82$

Using Equation (2-7):

$U_A = R_T\ U'_A$

$U_A = (0.82)\ (62.40) = 51.17$ knots

(6) Determine H_s, T_p, and t:

From Figure 48 for $U_A = 51.17$ knots and F = 35 nautical miles:

$H_s = 11.4$ feet

$T_p = 7.4$ seconds

t = 4.4 hours

THEREFORE: Assumed value of t = 4.4 hours was a good value and no further iteration is required. If the duration differed considerably then the new value of t from Figure 48 would be assumed for the duration of the wind and the process would be repeated until the answer converged.

EXAMPLE PROBLEM 10

Given:
a. Fetch interval, F_1 = 90,000 feet with d_1 = 35 feet
b. Fetch interval, F_2 = 60,000 feet with d_2 = 25 feet
c. Fetch interval, F_3 = 60,000 feet with d_3 = 20 feet
d. Adjusted windspeed, U_A = 40 miles per hour

Find: The significant wave height, H_s, at the end of fetch interval,

Solution: For fetch F_1 = 90,000 feet, U_A = 40 miles per hour, and d_1 = 35 feet, use Figure 55:

H_{s_1} = 4.2 feet at the end of F_1

Entering Figure 53 (for d_2 = 25 feet) with U_A = 40 miles per hour and H_s = 4.2 feet:

F'_1 = 130,000 feet

To obtain H_{s_2} at the end of F_2, enter Figure 53 (for d_2

= 25 feet) with U_A = 40 miles per hour and F = F'_1 + F_2

= 130,000 + 60,000 = 190,000 feet:

H_{s_2} = 4.5 feet at the end of F_2

Entering Figure 52 (for d_3 = 20 feet) with U_A = 40 miles per hour and H_s = 4.5 feet:

F'_2 = 420,000 feet

To obtain H_{s_3} at the end of F_3, enter Figure 52 (for d_3

= 20 feet) with U_A = 40 miles per hour and F = F'_2 + F_3

= 420,000 + 60,000 = 480,000 feet

H_s = 4.5 feet at the end of F_3

4. SOURCES FOR WAVE OBSERVATION DATA. The U.S. Naval Weather Service Command conducts a program to publish a Summary of Synoptic Meteorological Observations (SSMO) based upon shipborne observations. The observations are given over a specified time period, usually 10 to 30 years, and within a certain geographical area. Data are presented in tables. Table 18 of the SSMO gives percent frequency of occurrence of wave height by season and direction as a function of windspeed. Table 19 of the SSMO gives percent frequency of occurrence of wave period as a function of wave height. Wave distributions can be estimated by use of these tables for large geographic areas covering grids of several degrees. The tables give observed values which are assumed to be significant heights and periods. However, caution should be exercised

because ships may avoid waters with high waves and reporters often overlook a swell condition when a local wind wave obscures the swell. Data sources are as follows:

(1) In areas where heavy shipping occurs, data may be requested in specific grids of 1 degree or other larger- or smaller-degree grids at the National Climatic Center, Federal Building, Asheville, North Carolina 28801.

(2) Another source of wave data is Ocean Wave Statistics by Hogben and Lumb, London, Her Majesty's Stationery Office, 1967.

(3) The U.S. Navy Fleet Numerical Weather Central in Monterey, California, may be consulted to obtain wave hindcasts for specific stations where data have been compiled.

(4) Other sources may include results of wave-gage analyses, as well as special reports and studies of hindcasts for specific locations. These must be obtained through a local source or through a search of available literature.

5. EXTREME WAVES. Selection of the design wave either requires proof that the wave height is limited by water depth or an analysis to determine the frequency of occurrence of waves in deeper water. Generally, wave-gage data sets are limited to 1 to 3 years of data. Shipborne observations may cover a 10- to 30-year range. Synoptic charts of extreme storm events may cover a 20- to 40-year period or more. The recurrence interval, or period of time that a given wave height should be exceeded based on statistics of past observations, is required in order to select the design wave and estimate damages if that wave height be exceeded. The normal procedure is to determine the percent frequency that the wave heights in the data set are exceeded. These data are typically plotted on, for example, lognormal, semilog, log-probability, or normal-probability graph papers. The percent frequency of exceedence must then be related to a recurrence interval In years. This is easily done if the maximum storm every year is known, or if a set of extreme storms in a given period is given. An additional assumption is required if the data are given in hours or percent occurrence per year. Normal design procedures use from 3 to 12 hours duration per year for the annual significant wave. The choice depends upon judging the factors of minimum duration that are required to develop a fully arisen sea for the design wind and fetch, the frequency of observations, and the consequences of damage if a slightly lower design wave is selected.

EXAMPLE PROBLEM 11

Given: Summary of wave statistics for all directions as shown in Table 2.

Find: Draw frequency of exceedence curve for wave height and determine design 20-, 50-, and 100-year wave heights for all directions.

Solution: Beginning with the second highest wave class (8 to 9 feet), subtract the cumulative total (99.96) of this wave class from 100. Plot this value (0.04) on semilogarithmic

EXAMPLE PROBLEM 11 (Continued)

TABLE 2
Percent Frequency of Occurrence for Example Problem 11
(10 years of data; 1,825 observations)

Wave Height (feet)..	0-1	2-3	4-5	6-7	8-9	10
Direction						
N.............	2.50	1.63	0.90	0.01	0.01	0.00
NE............	5.10	3.20	2.00	0.05	0.01	0.01
E.............	8.98	6.51	1.30	0.54	0.12	0.02
SE...........	11.22	8.10	1.30	0.03	0.03	0.01
S............	14.61	10.81	0.10	0.02	0.02	0.00
SW............	3.92	4.82	0.20	0.10	0.01	0.00
W.............	3.46	4.00	0.30	0.20	0.00	0.00
NW............	2.01	1.33	0.50	0.01	0.00	0.00
Total	51.80	40.40	6.60	0.96	0.20	0.04
Cumulative Total	51.80	92.20	98.80	99.76	99.96	100.00

paper at the highest wave in that class (H = 9 feet). For the next wave class (6 to 7 feet), follow the same procedure and plot this value (0.24) at H = 7 feet. Continue for all wave classes, and then draw the best possible line through the points. This line, shown in Figure 59, is the frequency of exceedence curve.

To determine design wave heights, begin by extrapolating the frequency of exceedence curve back. The following percentages of occurrence for 20-, 50-, and 100-year storms were calculated (assuming a duration of 12 hours for a storm):

$$\text{20-year: } \left[\frac{12 \text{ hours}}{\left(365 \frac{\text{days}}{\text{year}}\right)\left(24 \frac{\text{hours}}{\text{day}}\right)(20 \text{ years})} \right](100) = 0.00685\%$$

$$\text{50-year: } \left[\frac{12 \text{ hours}}{\left(365 \frac{\text{days}}{\text{year}}\right)\left(24 \frac{\text{hours}}{\text{day}}\right)(50 \text{ years})} \right](100) = 0.00274\%$$

$$\text{100-year: } \left[\frac{12 \text{ hours}}{\left(365 \frac{\text{days}}{\text{year}}\right)\left(24 \frac{\text{hours}}{\text{day}}\right)(100 \text{ years})} \right](100) = 0.00137\%$$

The 20-, 50-, and 100-year design wave heights can be found directly from the frequency of exceedence curve by reading the wave height at the respective percentage of occurrence. Thus the 20-, 50-, and 100-year design wave

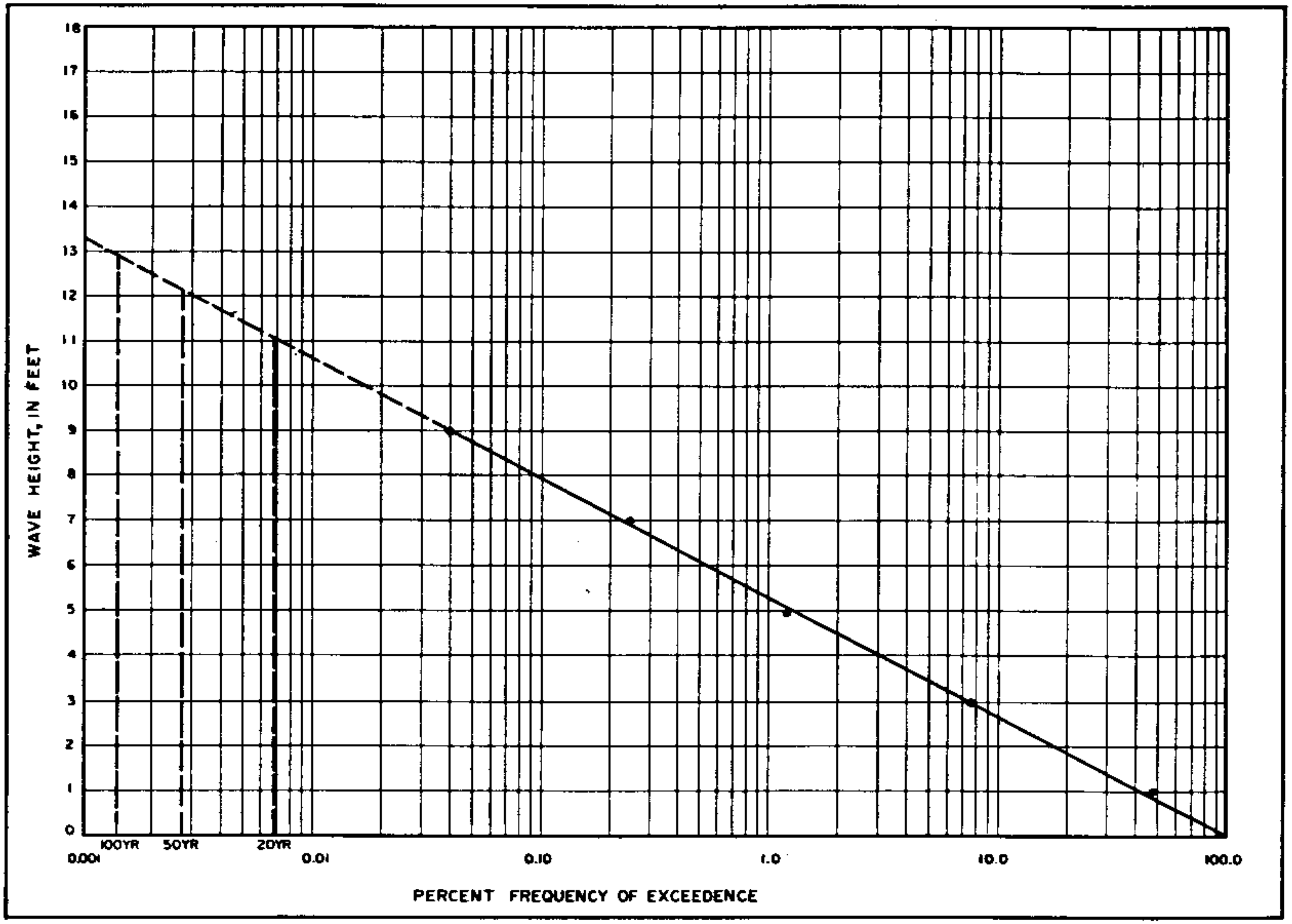


FIGURE 59
Frequency of Exceedence Curve, and 20-, 50-, and 100-Year Design Wave Heights, for Example Problem 11

Design Wave Heights, for Example Problem 11] heights are 11.1, 12.2, and 12.9 feet, respectively. (See Figure 59.) It should be noted that in areas of typhoon or hurricane activity, more detailed studies may be required.

6. SELECTION OF DESIGN WAVES.

a. Selection. The selection of design waves should be related to the economics of construction, maintenance, and repairs. For small projects, a 20- to 25-year design wave, coupled with an annual extreme water level, is appropriate. In special cases, such as over a coral reef or in the breaker zone, the design water level may control the design wave height. (See DM-26.1, Section 2.7, for a discussion of design water level.)

b. Large Projects. The selection of design conditions for larger structures requires more detailed consideration of the economics of the design. Wave analysis yields the recurrence interval of a given wave height. If, for example, the design wave height having a recurrence interval of 20 years is 10 feet, then a wave having a 30-year recurrence interval and height of 15 feet will damage the structure. The economics of increasing the first cost versus making occasional repairs must be evaluated. Furthermore, cost and extent of damages to areas that the structure is designed to protect must also be considered. The physical and economic factors, such as design wave height versus annual costs, must be optimized.

The principal of optimization is schematically shown in Figure 60, where annual cost is plotted as a function of design wave height. The plot is made by designing the structure for a range of wave heights. As the design wave height increases, first cost of the structure increases. (The first cost must be related to an annual cost. This is accomplished by amortizing the first cost by using an appropriate interest rate and time period.) The annual maintenance cost will decrease if the structure is designed for a larger wave. This curve is difficult to plot accurately because several arbitrary decisions must be made concerning how many times in the life of the structure repairs must be made and how much maintenance costs would be. By making reasonable assumptions, or at least by incorporating the principal of optimization into the design, a selection of the design wave can be made by adding the annual maintenance cost and the annual first cost to produce a third curve which represents the annual cost of the structure. The designer then can identify the wave height which represents the least annual cost. Some of the decisions involved in arriving at this optimum design wave height are arbitrary and not based on hard data regarding maintenance costs; therefore, some latitude should be permitted in the selection of the design wave height using the optimization procedure. If the cost varies 5 to 10 percent, the optimum design wave would have a range of heights. The designer should use other factors to help select the proper design condition, such as environmental, operational, and maintenance considerations.

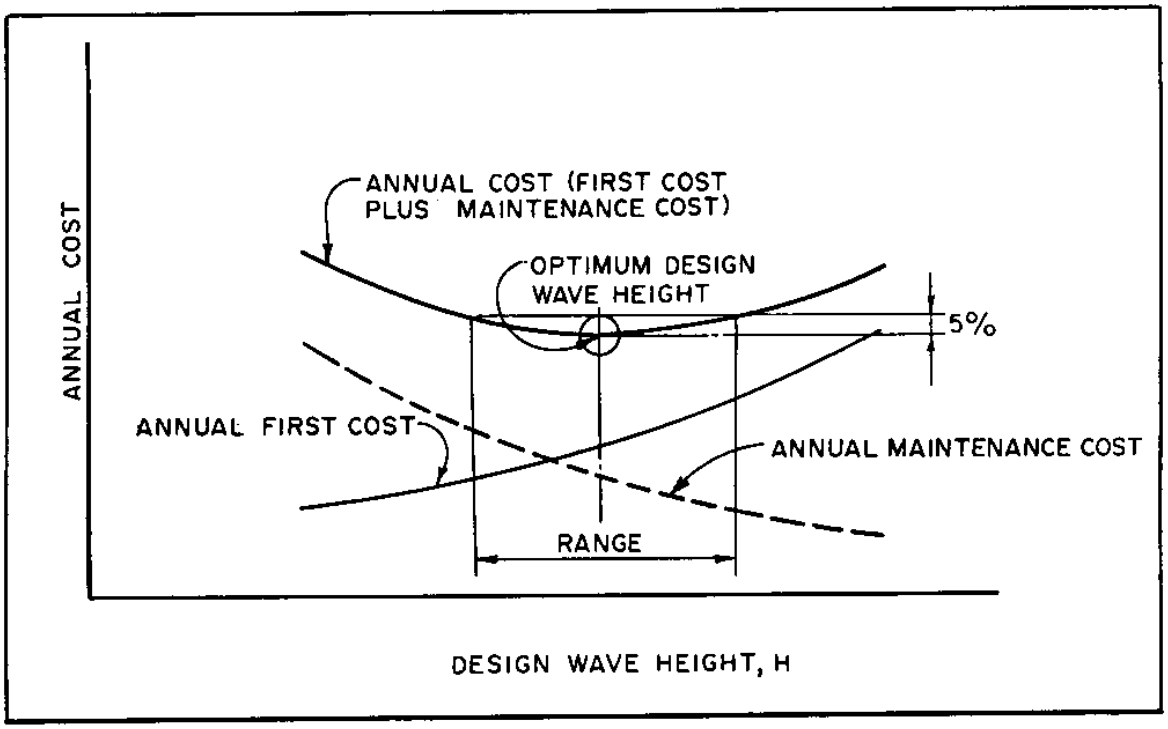


FIGURE 60
Selection of Optimum Design Wave Height

This procedure need not be strictly employed; however, the designer should consider the principles involved in selecting the optimum design. Such an analysis should prevent selection of a 1,000-year return period typhoon wave for design of a small boat harbor, or a 1-year return period sea for design of a cargo-wharf piling system.

c. Wave-Height Variability. Most wave-transformation studies calculate the significant wave height, H_s, at the project site. Wave systems have a wave-height distribution where the significant height is exceeded. The rigidity of the structure, or its ability to withstand an occasional larger wave, must be evaluated. Table 3 summarizes general guidance for selecting the appropriate wave height.

TABLE 3

General Factors For Wave-Height Selection

Type of Structure	Wave Height[1]	Example
Nonrigid: minor damage to armor units can be tolerated without threat to the function of the structure	H_s	Rubble-mound breakwaters and revetments; pile-supported structures
Semirigid: the structure can absorb some excessive wave force without catastrophic failure	H_{10}	Cellular sheet-pile walls
Rigid: damage may cause complete failure if the design wave is slightly exceeded	H_1	Cantilever sheet-pile walls; braced sheet-pile walls

[1]Wave heights are defined in Section 2.2.a., Significant Wave Height.

7. METRIC EQUIVALENCE CHART. The following metric equivalents were developed in accordance with ASTM E-621. These units are listed in the sequence in which they appear in the text of Section 2. Conversions are approximate.

32.8 feet = 10 meters
10 miles = 16.1 kilometers
50 feet = 15.2 meters
10 feet = 3.0 meters
15 feet = 4.6 meters

SECTION 3. BASIC PLANNING

1. GENERAL. The type of structure required for a particular design situation depends upon the protection required, such as harbor protection, beach erosion control, and stabilization of an entrance channel. Table 4 describes the primary types of coastal structures and their functions. In many cases, more than one type of structure may provide a possible solution. Studies of alternative solutions, including consideration of first and annual costs, maintenance, construction methods, and environmental impacts, should be conducted to select the most appropriate one. Figures 61 through 67 give examples of typical uses and construction of each structure type. Selection of the structure type requires that the foundation condition, availability of construction materials and equipment, and probable impacts on the adjacent shores be considered.

2. ENVIRONMENTAL CONSIDERATIONS.

a. Discussion. The Coastal Zone Management (CZM) Act of 1972, PL 92-583, establishes a national policy to preserve, protect, develop, and, where possible, restore and enhance the resources of the coastal zone of the United States. DOD Instruction 4165.59 of 29 December 1975 authorized the Navy to implement programs to achieve the objectives of PL 92-583. The Navy will cooperate and provide information on Navy programs within the coastal zone to states responsible for developing state CZM plans. Naval operations, activities, projects, or programs affecting coastal lands or waters shall insure that such undertakings, to the maximum extent practicable, comply with state-approved coastal-zone programs.

b. Guidelines and Standards.

(1) All natural resources management programs on naval installations in the coastal zone have potential effects on the coastal zone and should be reviewed for consistency with approved state Coastal Zone Management plans. The Navy shall develop, in cooperation with a designated state agency, a set of criteria and standards for judging the consistency of natural resource management programs with respect to approved state management programs. Consistency determinations shall be made in accordance with provisions of PL 92-583.

(2) Agricultural outlease of real property affecting land or water uses in the coastal zone shall provide a certification that the proposed use complies with the coastal state's approved program and that such usage will be conducted in a manner consistent with the program.

(3) Technical assistance requested by the states to assist their implementation of CZM will be provided to the extent practicable. Data collected by the Navy on subjects such as beach erosion, hydrology, meteorology, and navigation may be useful for coastal-zone planning and shall be made available.

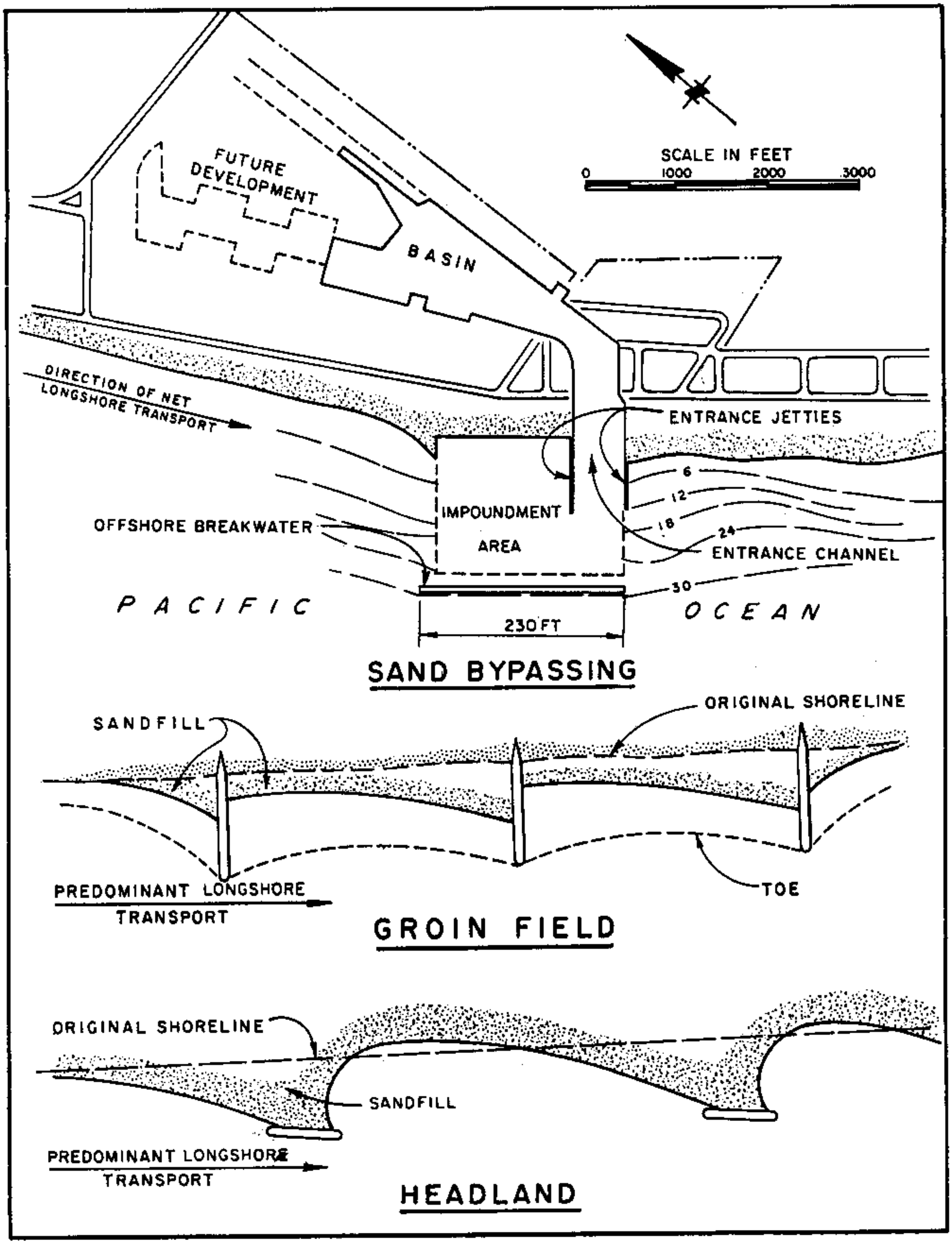


FIGURE 61
Typical Uses of Breakwaters, Jetties, Groins, and Artificial Headlands in Coastal Protection

Artificial Headlands in Coastal Protection]

26.2-88

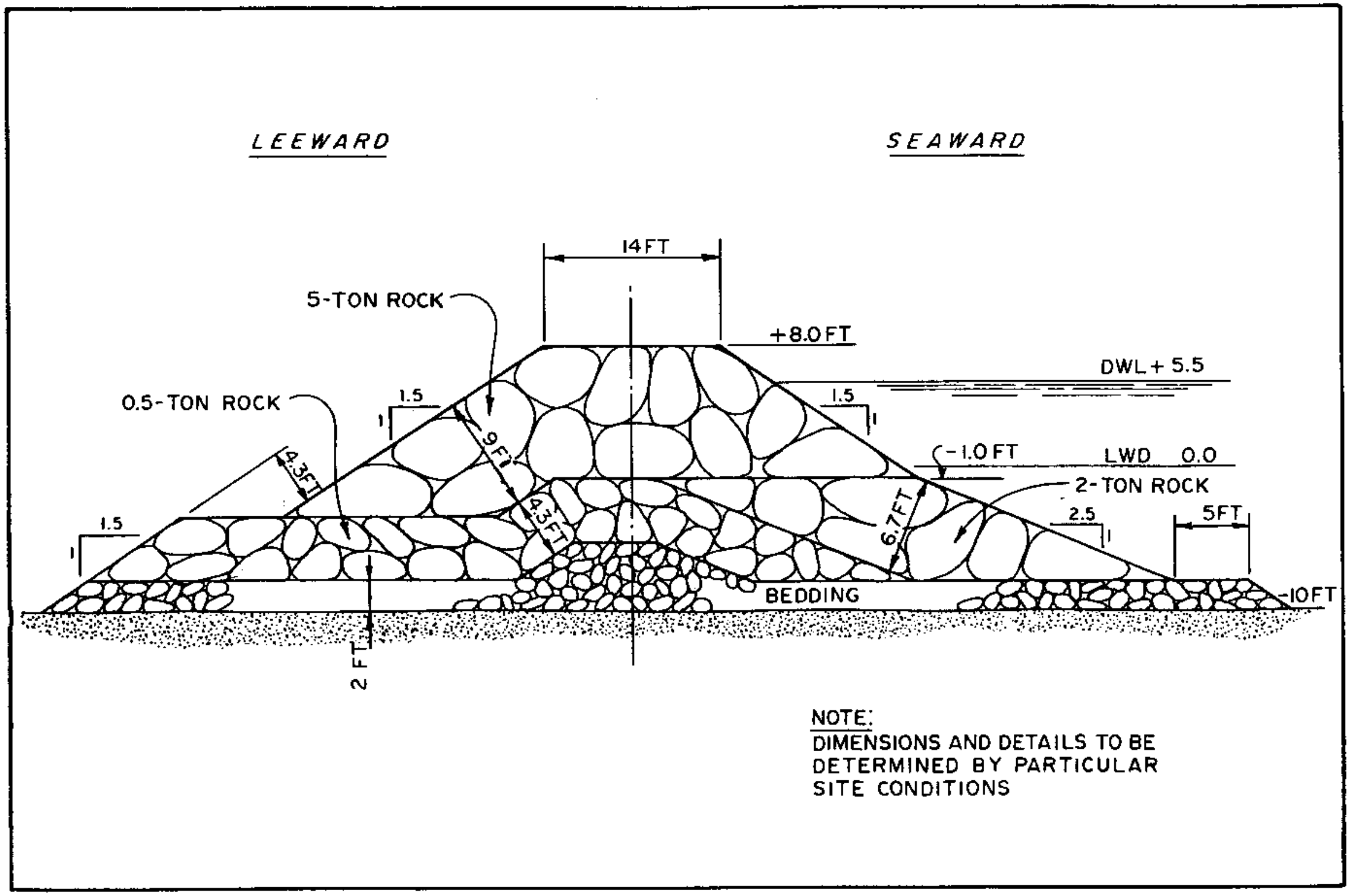


FIGURE 62
Typical Breakwater Section

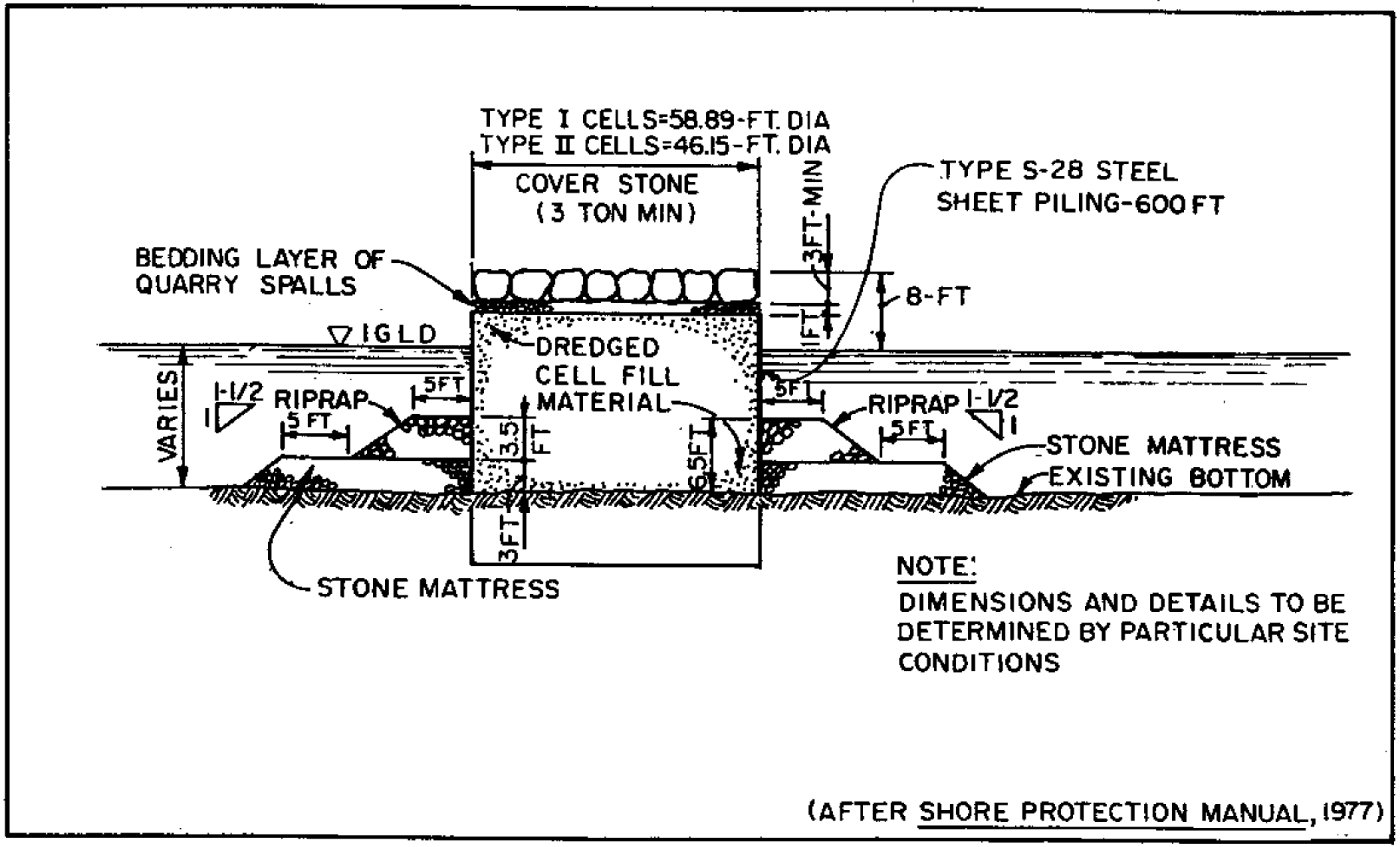


FIGURE 63
Typical Cellular Sheet-Pile Jetty

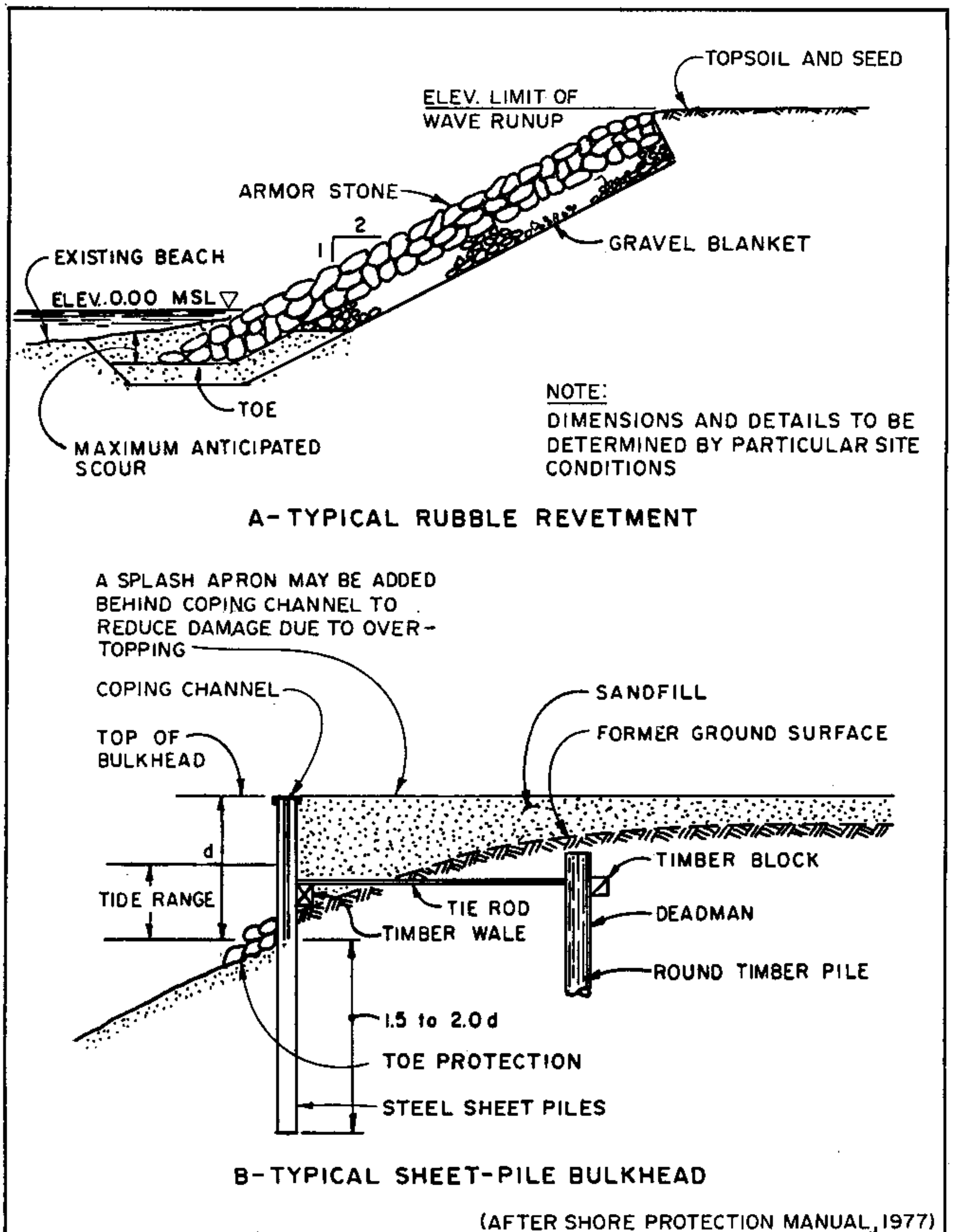


FIGURE 64
Typical Rubble Revetment and Typical Sheet-Pile Bulkhead

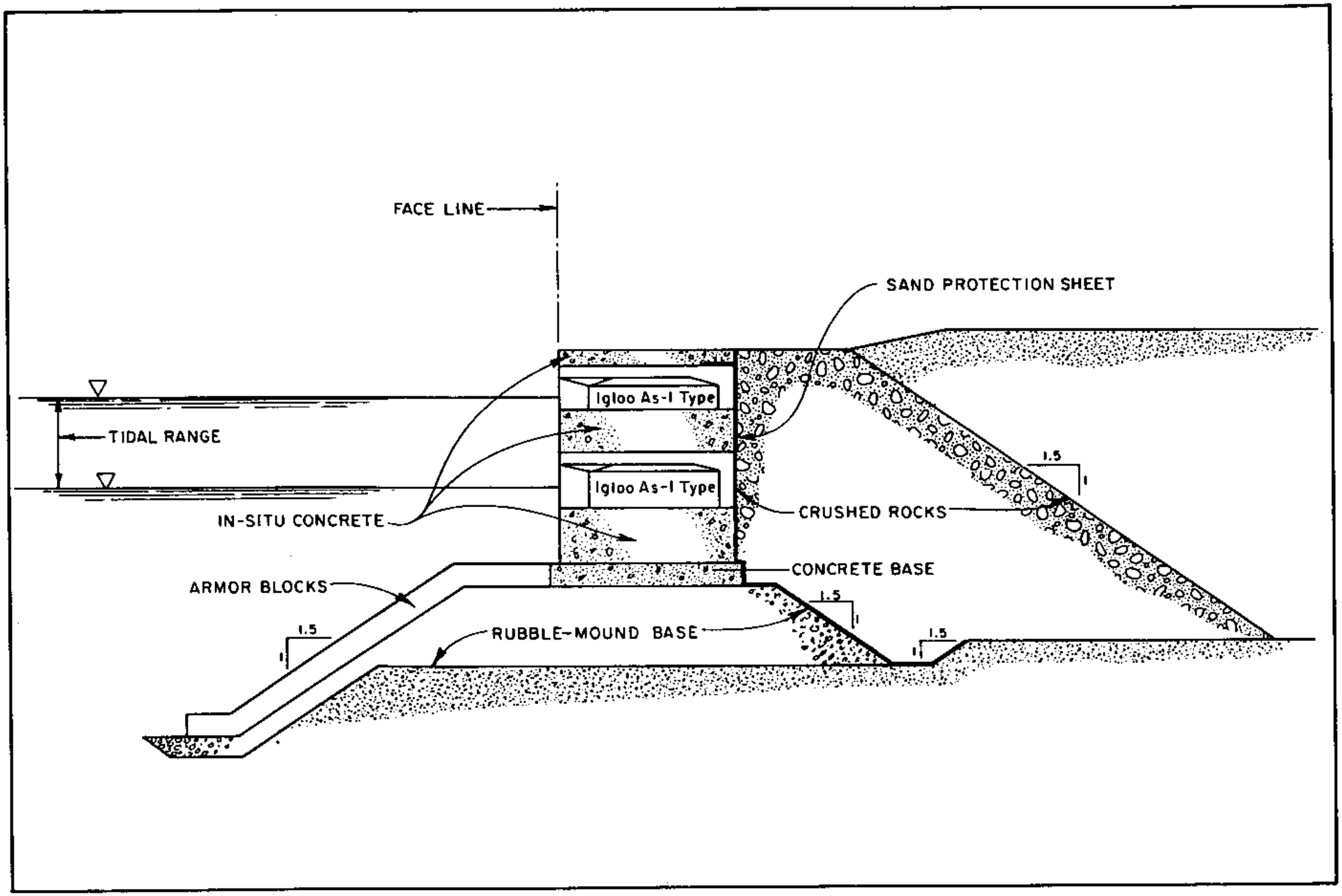


FIGURE 65
Typical Igloo Seawall

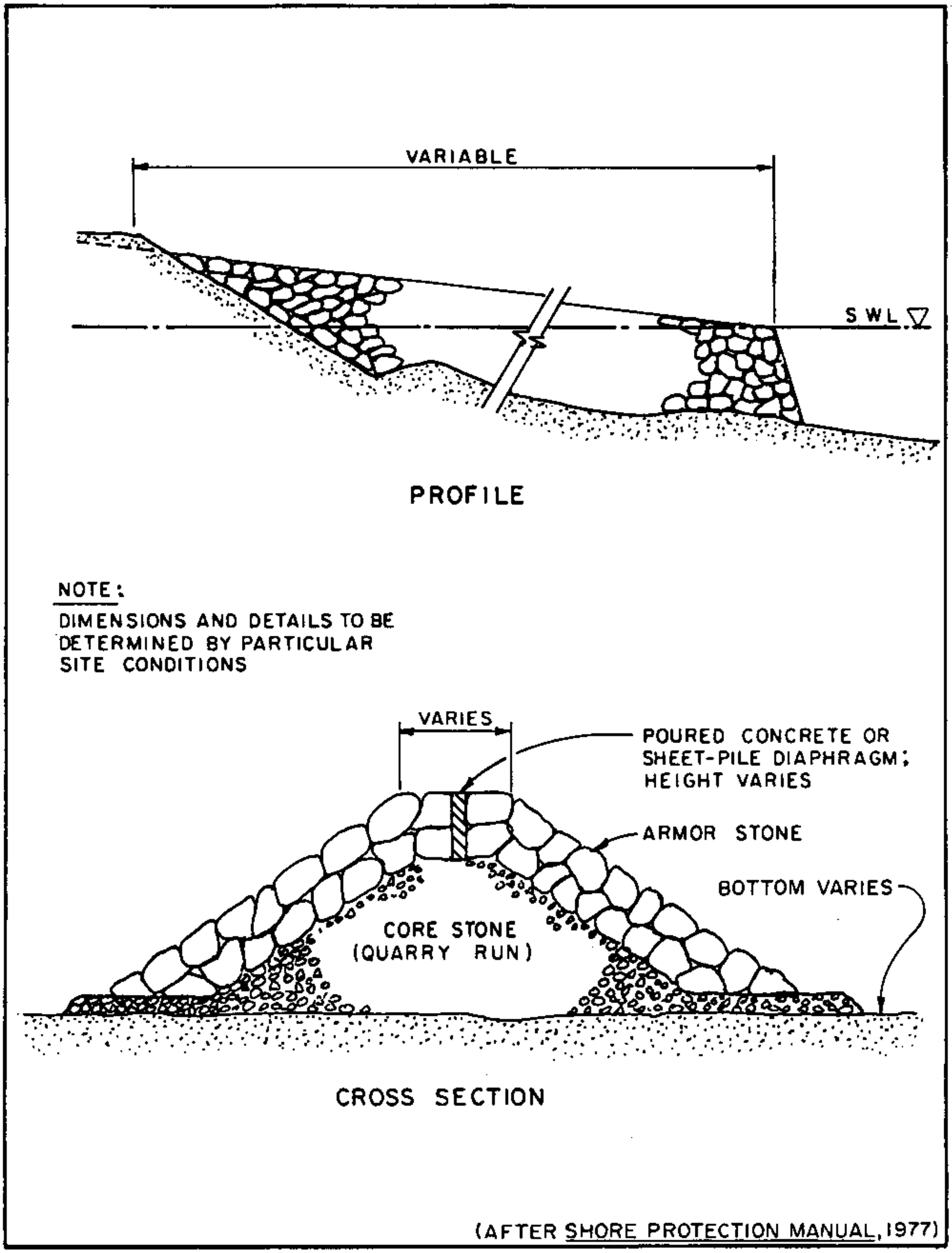


FIGURE 66
Typical Rubble-Mound Groin

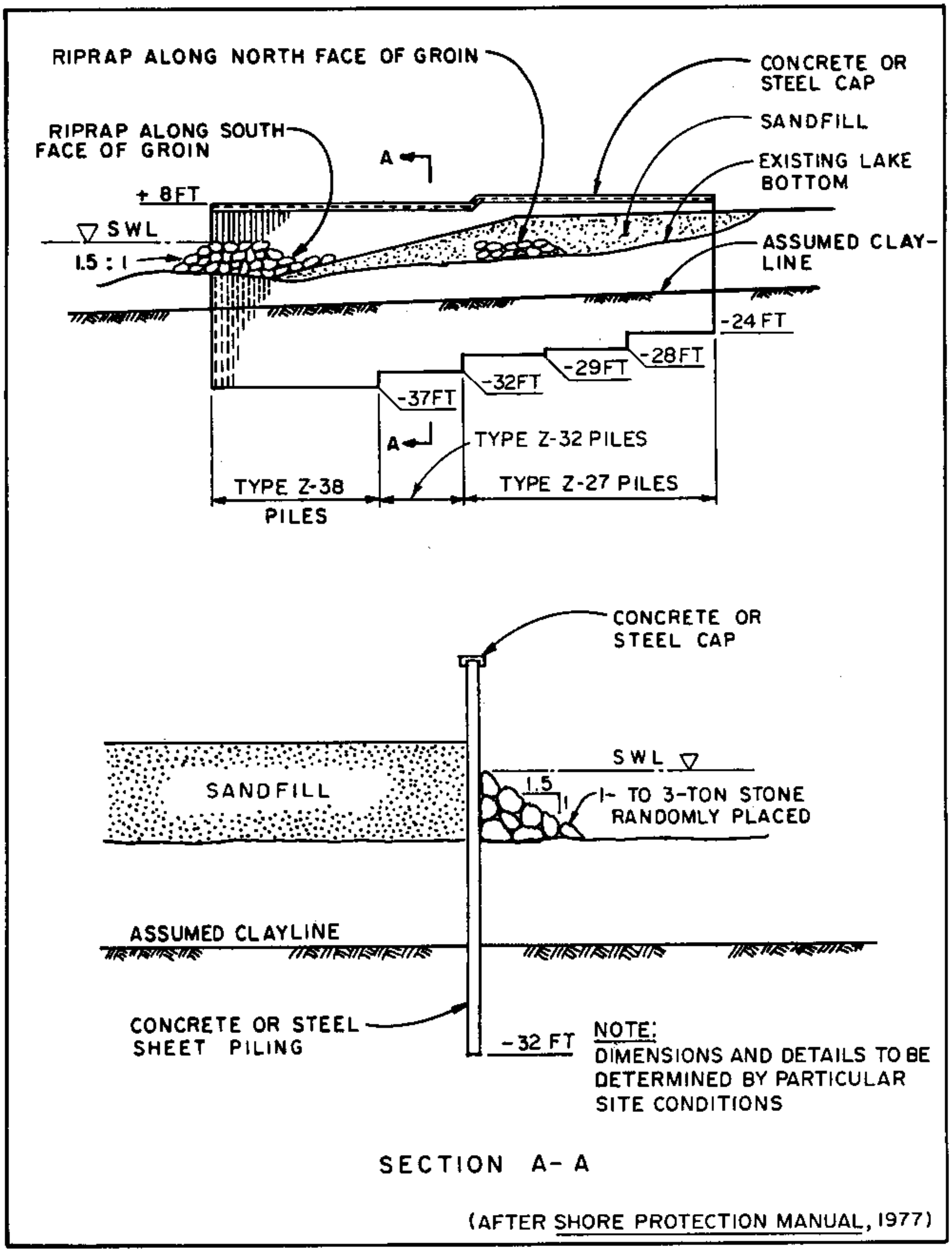


FIGURE 67
Typical Sheet-Pile Groin

TABLE 4
Primary Types of Coastal Structures

Structure	Function
Breakwaters	Primary applications of breakwaters are to provide protection against waves for shore areas, harbors, anchorages, and basins, and to enable maintenance-dredging operations. A secondary purpose is beach erosion control.
Jetties	Jetties are devices parallel to a navigation channel used to protect the channel from shoaling with littoral drift and to stabilize the entrance to a tidal inlet. They may also provide wave and wind protection and direct or confine the flow of river or tidal currents. Sand bypassing of jettied inlets is often necessary to preclude erosion of the downdrift coast.
Revetments, bulkheads, and seawalls	These structures are used to protect embankments or shore structures from eroding or from damage due to wave attack or currents and to retain or prevent sliding of land. Revetments are generally rubble construction. Seawalls and bulkheads are generally more rigid structures constructed of steel, concrete, or timber. Another design for seawalls is to use Igloos, patented by Nippon Tetrapod (see Figure 65). Igloos can be used as space-saving wave absorbers or breakwaters. Prefabricated concrete units have been used successfully as wave-dissipating walls in harbors.
Groins	Groins are used to protect the coast from erosion and to retard or control littoral transport to stabilize a beach. Groin fields should generally be filled with imported material to preclude erosion of the downdrift coast.
Headlands	Headlands are high, steep-faced border points of land extending into the ocean or other body of water. Large segments of shorelines can be stabilized by construction of artificial head-lands.
Beach restoration and nourishment.	Beaches that are eroding due to an interrupted or inadequate sand source can be stabilized by deposition of sand brought from a source on land or of dredged materials.

ÄÄ

3. FUNCTIONAL DESIGN. To be effective, a breakwater must be built to a high enough elevation and be impermeable to the extent that waves transmitted to the lee side are attenuated to acceptable levels. Wave transmission studies are required to determine appropriate crest elevations. If design criteria stipulate that a breakwater or revetment is not to be overtopped, wave-runup studies are required. Wave-runup calculations, coupled with observations at neighboring structures and beaches, are required to determine the crest elevation of a structure or the berm elevation of a protective beach. If wave overtopping can cause flooding or undesirable ponding of water, calculations of overtopping quantities are required. Methods for calculating wave runup and transmitted-wave heights are presented herein. Calculation of overtopping quantities is rarely needed; the reader is referred to the Shore Protection Manual (1977) for details on the proper procedure for such calculations.

4. WAVE RUNUP.

a. Definition. Wave runup, R, is the vertical height above the still water level to which water from an incident wave reaches when it encounters a structure or natural formation such as a beach. If the structure is lower in height than the runup elevation, the structure is overtopped. Figure 68 is a sketch defining runup and overtopping terms. Runup is a function of the characteristics of the wave structure and of the offshore slope. Runup on coastal structures can be calculated from small-scale model studies; however, adjustments may be necessary to account for model-to-prototype scale effects and for structure-roughness effects.

b. Calculation of Runup.

(1) General. The calculation of wave runup is based on the results of small-scale hydraulic model studies. m e model studies were done for special cases of structures on horizontal bottoms, sloping bottoms, embankments or revetments, breakwaters with low, medium, or high impermeable cores, and with quarrystone or concrete armor units. Runup depends on: relative depth at the toe of the structure, wave steepness, structure slope, beach slope, roughness of the structure, and relative core heights. The equivalent unrefracted deepwater wave height, H'_o, is used in all the runup procedures given below, except for vertical walls subjected to nonbreaking or nonbroken waves. (In such a case, the incident wave height, H_i, is used.) Runup, R, is given by:

$$R = (H'_o)(R/H'_o)(r)(k) \qquad (3\text{-}1)$$

WHERE: R = runup

H'_o = equivalent unrefracted deepwater wave height

R/H'_o = relative runup

r = rough-slope runup correction factor

k = runup scale-effect correction factor

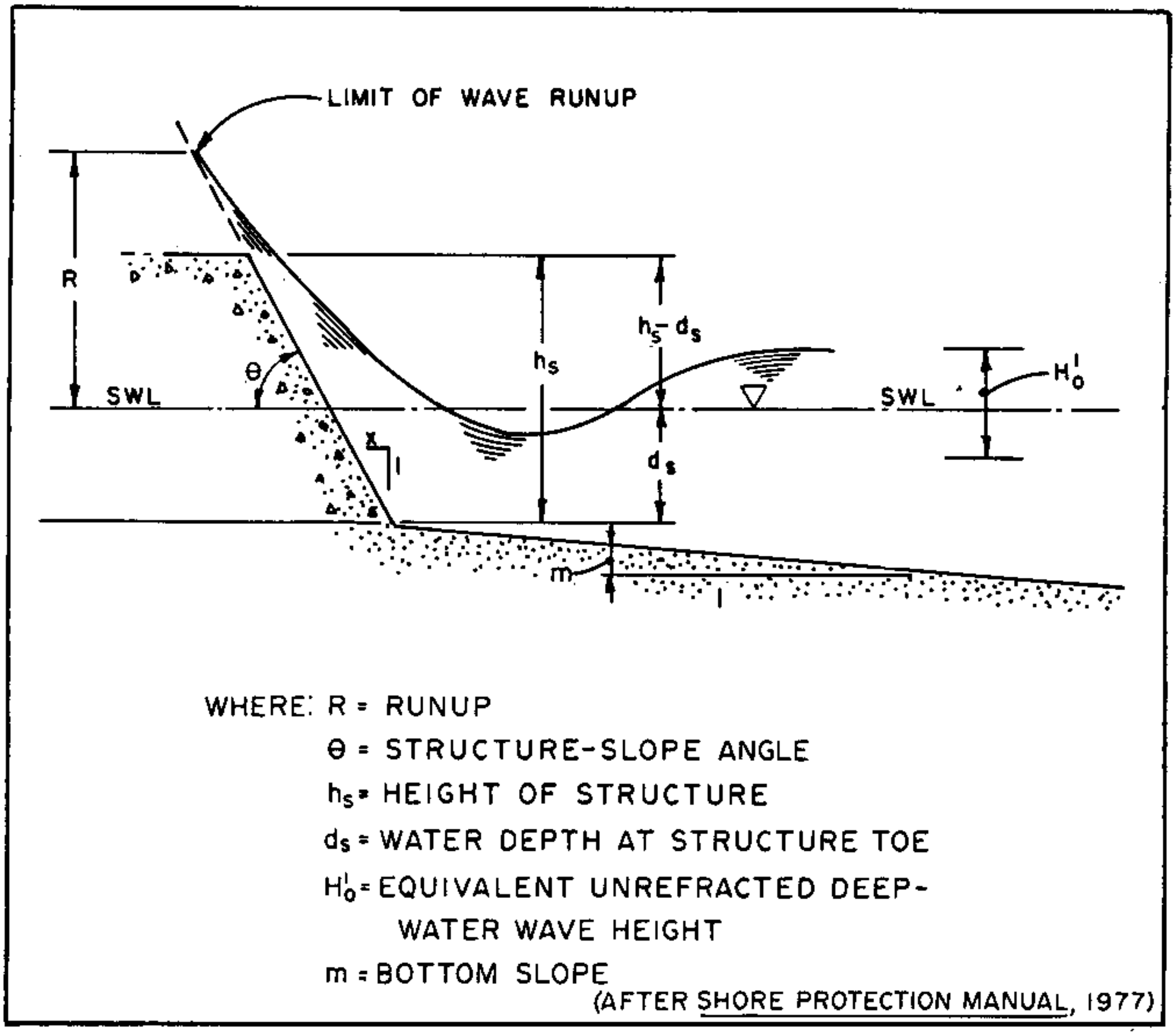


FIGURE 68
Definition of Runup and Overtopping Terms

Runup, R, is the distance above the given water level. The actual runup elevation is determined by adding the runup, R, to the water level used in the calculation.

In the subsections which follow, procedures are described for calculating runup on different types of structures. These procedures were derived from Stoa (1979). Table 5 summarizes which procedure to follow for a given situation.

TABLE 5
Wave-Runup Procedures

Structure Type	Armor Type	Case	Subsection
Embankment or revetment.	Quarrystone	1	3.4.b.(2)
Embankment or revetment.	Concrete	2	3.4.b.(3)
Breakwater	Rubble-mound		
	Low core	3	3.4.b.(4)(a)
	Medium core	4	3.4.b.(4)(b)
	High core	5	3.4.b.(4)(c)
Breakwater	Concrete--all co	6	3.4.b.(5)
Vertical structures. . .	Solid	7	3.4.b.(6)
Beaches.	Sand to cobble	8	3.4.b.(7)

(2) Case 1: Embankment or Revetment, Quarrystone Armor. Wave runup on an embankment or revetment with quarrystone armor is determined by first finding the relative runup, R/H'_o, from Figures 69-81. The figure to be used depends upon the slope fronting the structure, cot [theta], and upon the value of d_s/H'_o. Figures 69-71 are "rough-slope" curves, whereas Figures 72-81 are "smooth-slope" curves. To use these "smooth-slope" curves in determining runup on the rough slope of a quarrystone embankment or revetment, the rough-slope runup correction factor, r, is applied.

(a) Structure fronted by horizontal bottom; d_s/H'_o > / = 3 and/or 1.5 < / = cot [theta] < / = 5. Find the relative runup, R/H'_o, as a function of the cotangent of the structure slope, cot [theta], and of the deepwater wave steepness, H'_o/gT^2, from Figures 69, 70, or 71, depending on relative depth, d_s/H'_o. If d_s/H'_o = 3.0, use Figure 69. If d_s/H'_o = 5.0, use Figure 70. If d_s/H'_o > / = 8.0, use Figure 71. The rough-slope runup correction factor, r, and the runup scale-effect correction factor, k, are both unity. Then the runup is:

$$R = (H'_o)(R/H'_o)(r)(k)$$

$$R = (H'_o)(R/H'_o) \qquad (3\text{-}2)$$

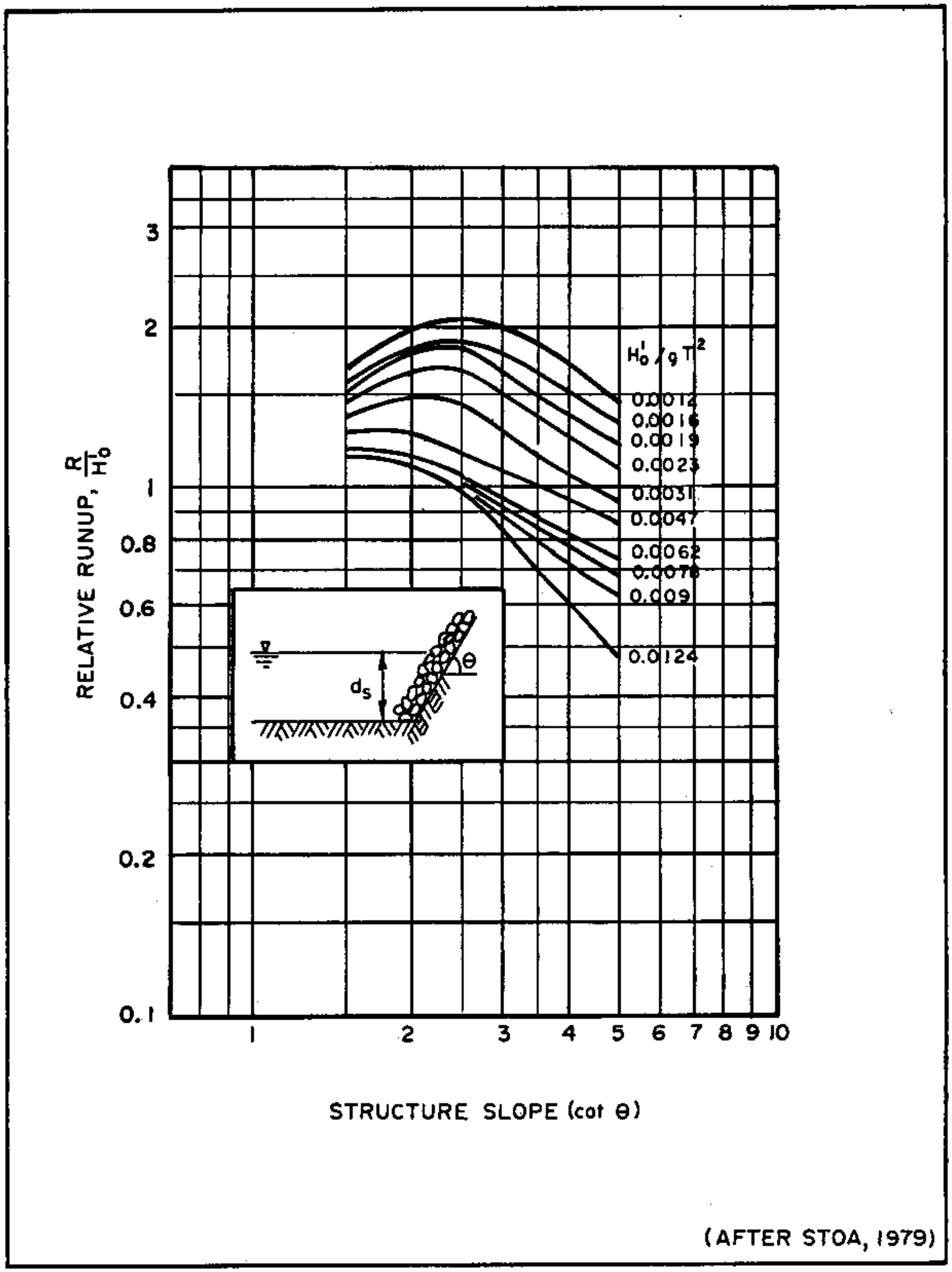


FIGURE 69
Relative Runup, R/H_o', on a Rough Embankment or Revetment for Relative Depth, $d_s/H_o' = 3.0$

Revetment for Relative Depth, dÚs¿/H'Úo¿ = 3.0]

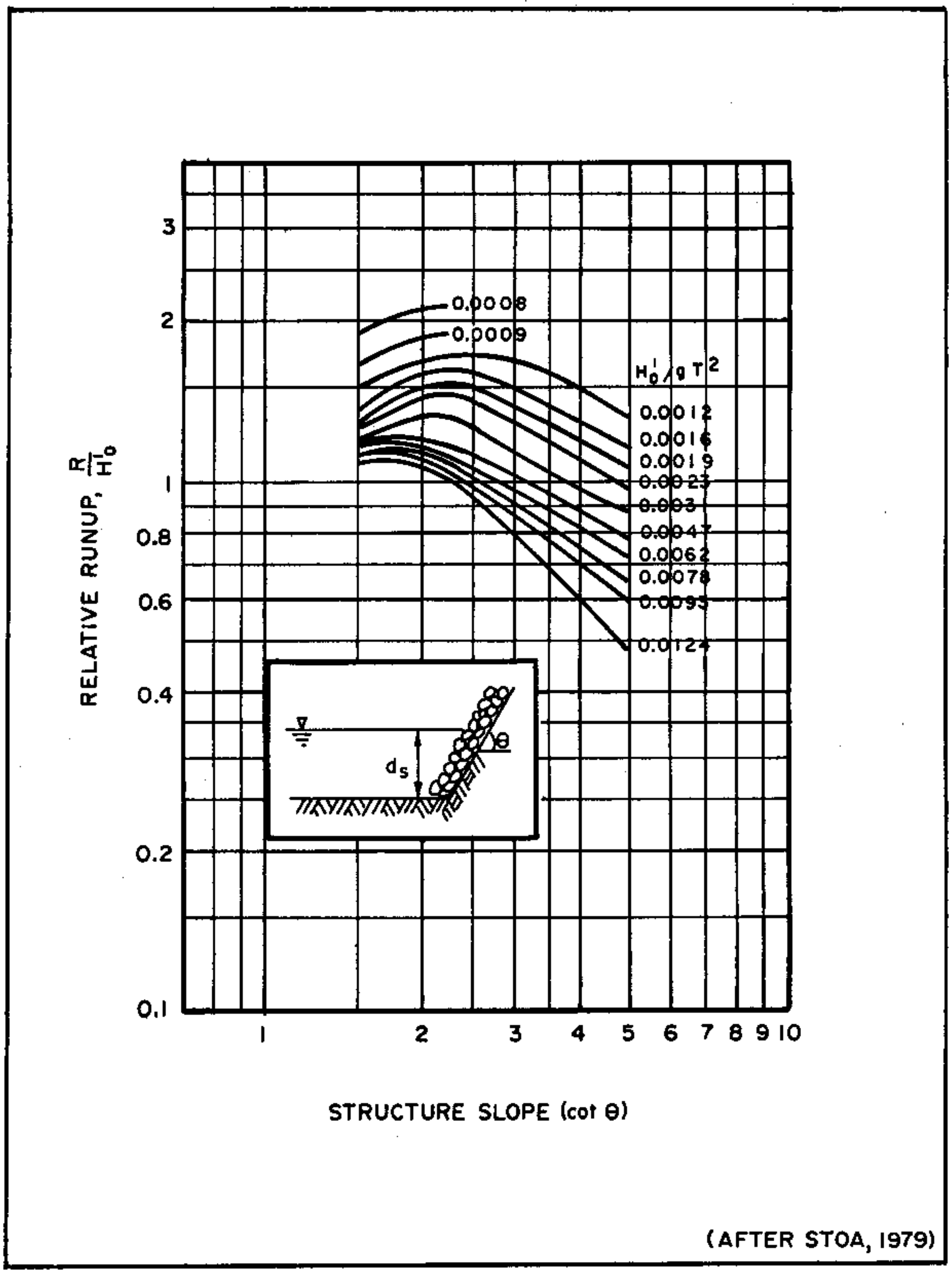


FIGURE 70

Relative Runup, R/H_o', on a Rough Embankment or Revetment for Relative Depth, $d_s/H_o' = 5.0$

Revetment for Relative Depth, dÚs¿/H'Úo¿ = 5.0]

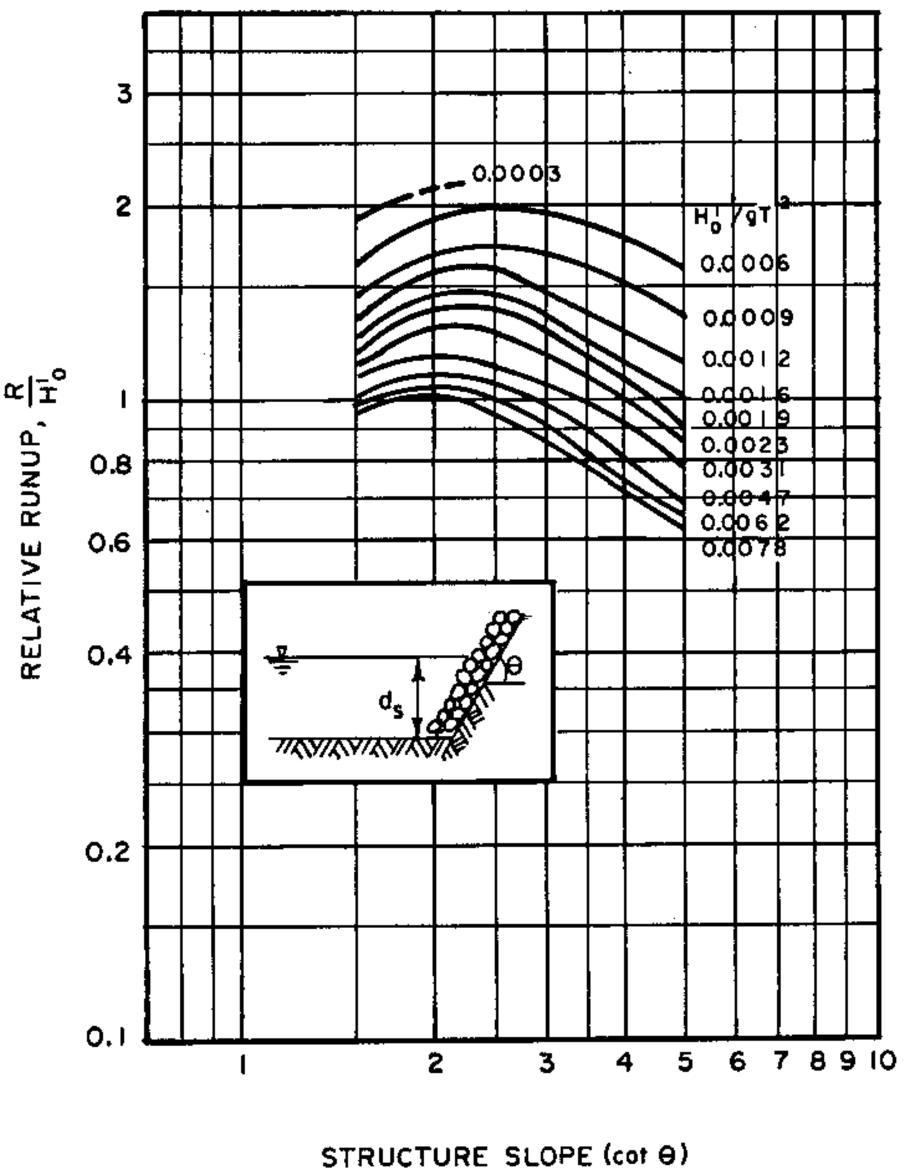


FIGURE 71
Relative Runup, R/H'_o, on a Rough Embankment or Revetment for Relative Depth, $d_s/H'_o \geq 8.0$

Revetment for Relative Depth, dÚs¿/H'Úo¿ > / = 8.0]

EXAMPLE PROBLEM 12

Given: a. The equivalent unrefracted deepwater wave height, H'_o = 10 feet
b. Water depth at structure toe, d_s = 30 feet
c. Wave period, T = 8 seconds
d. Structure slope, cot [theta] = 1.5

Find: Runup for a quarrystone revetment.

Solution: (1) Find d_s/H'_o:

$$\frac{d_s}{H'_o} = \frac{30}{10} = 3; \text{ therefore, use Figure 69}$$

(2) Find $H'_o/g T^2$

$$\frac{H'_o}{g T^2} = \frac{10}{(32.2)(8)^2} = 0.0049$$

(3) From Figure 69, for cot [theta] = 1.5 and $H'_o/g T^2$ = 0.0049:

$$\frac{R}{H'_o} = 1.26$$

(4) Using Equation (3-2), find R:

$$R = (H'_o)(R/H'_o)$$

$$R = (10)\ (1.26) = 12.6 \text{ feet}$$

R = 12.6 feet

Note: To obtain the elevation of the structure required to prevent overtopping, add the value of runup, R, to the water level used in the calculation.

(b) Structure fronted by horizontal bottom; d_s/H'_o < 3 and/or 1.5 > / = cot [theta] > / = 5. Find the relative runup, R/H'_o, as a function of the cotangent of the structure slope, cot [theta], and of the deepwater wave steepness, H'_o /g T^2, from Figure 72 for d_s/H'_o < / = 3. The rough-slope runup correction factor, r, is 0. 60, and the runup scale-effect correction factor, k, is 1.00.

$$R = (H'_o)R/H'_o)(r)(k) \qquad (3\text{-}3)$$

$$R = (H'_o)\ (R/H'_o)\ (0.60)$$

(c) Structure fronted by 1:10 slope; d_s/H'_o < / = 3. If the structure is fronted by a 1:10 slope with the slope length, *l, equal to or greater than one-half of the wavelength, L, (*l > / = 0.5 L), where L is the

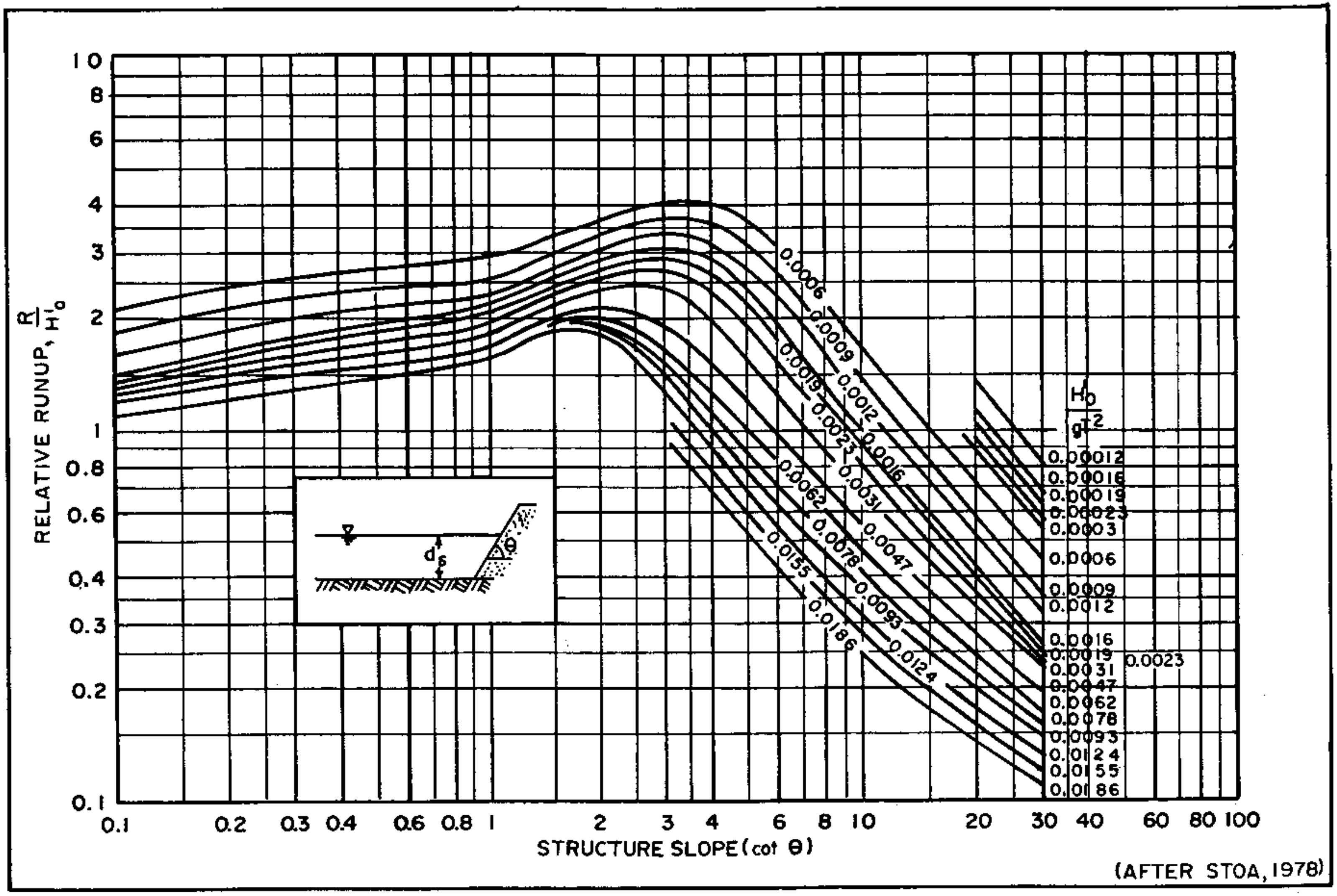


FIGURE 72

Relative Runup, R/H'_o, on a Smooth Embankment or Revetment for Relative Depth, $d_s/H'_o \leq 3.0$

Revetment for Relative Depth, dÚs¿/H'Úo¿ < / = 3.0] wavelength at the toe of the 1:10 slope, find the relative runup, R/H'Úo¿, as a function of the cotangent of the structure slope, cot [theta], and of the deepwater wave steepness, H'Úo¿/g TÀ2Ù, from Figures 73 through 76 for relative depths of dÚs¿/H'Úo¿ = 0.6, 1.0, 1.5, and 3, respectively. The rough-slope runup correction factor, r, is 0. 60, and the runup scale-effect correction factor, k, is 1.00; Equation (3-3) is used to calculate runup, R.

(d) Structure fronted by 1:10 slope; dÚs¿/H'Úo¿ > 3. For a relative depth of dÚs¿/H'Úo¿ > 3, the bottom is considered as horizontal and Figures 77 and 78 should be used. The rough-slope runup correction factor, r, is 0.60, and the runup scale-effect correction factor is 1.00; Equation (3-3) is used to calculate runup, R.

(e) Structure fronted by 1:10 slope; d = 0. If the toe of structure slope is at d = 0, then the relative runup, R/H'Úo¿, can be found in Figures 79, 80, or 81 for d/H'Úo¿ (rather than dÚs¿/H'Úo¿) equal to 3.0, 5.0, and 8.0, respectively; the depth, d, is taken as the depth at the toe of the 1:10 slope. The rough-slope runup correction factor, r, is 0.60, and the runup scale-effect correction factor, k, is 1.00; Equation (3-3) is used to calculate runup, R.

EXAMPLE PROBLEM 13

Given:
a. The wave height at the structure toe, H = 2. 75 feet
b. Water depth at structure toe, dÚs¿ = 6 feet
c. Wave period, T = 3 seconds
d. Structure slope, cot [theta] = 1.5
e. The bottom slope is horizontal.

Find: Runup for a quarrystone revetment.

Solution: (1) Find H'Úo¿:

LÚo¿ = (g/2[pi])TÀ2Ù = (32.2/2[pi])(3)= 46.1 feet

```
dÚs¿     6
ÄÄÄÄ = ÄÄÄÄ =  0.130
LÚo¿   46.1
```

From Figure 2 for dÚs¿/LÚo¿ = 0.13:

```
  H
ÄÄÄÄÄ = 0.92
H'Úo¿
```

```
         H     2.75
H'Úo¿ = ÄÄÄÄ = ÄÄÄÄ = 2.99; use 3.0 feet
        0.92   0.92
```

(2) Find dÚs¿/H'Úo¿:

```
dÚs¿   6
ÄÄÄÄ = Ä = 2; therefore, use Figure 72 (for dÚs¿/H'Úo¿ < / = 3.0)
H'Úo¿  3
```

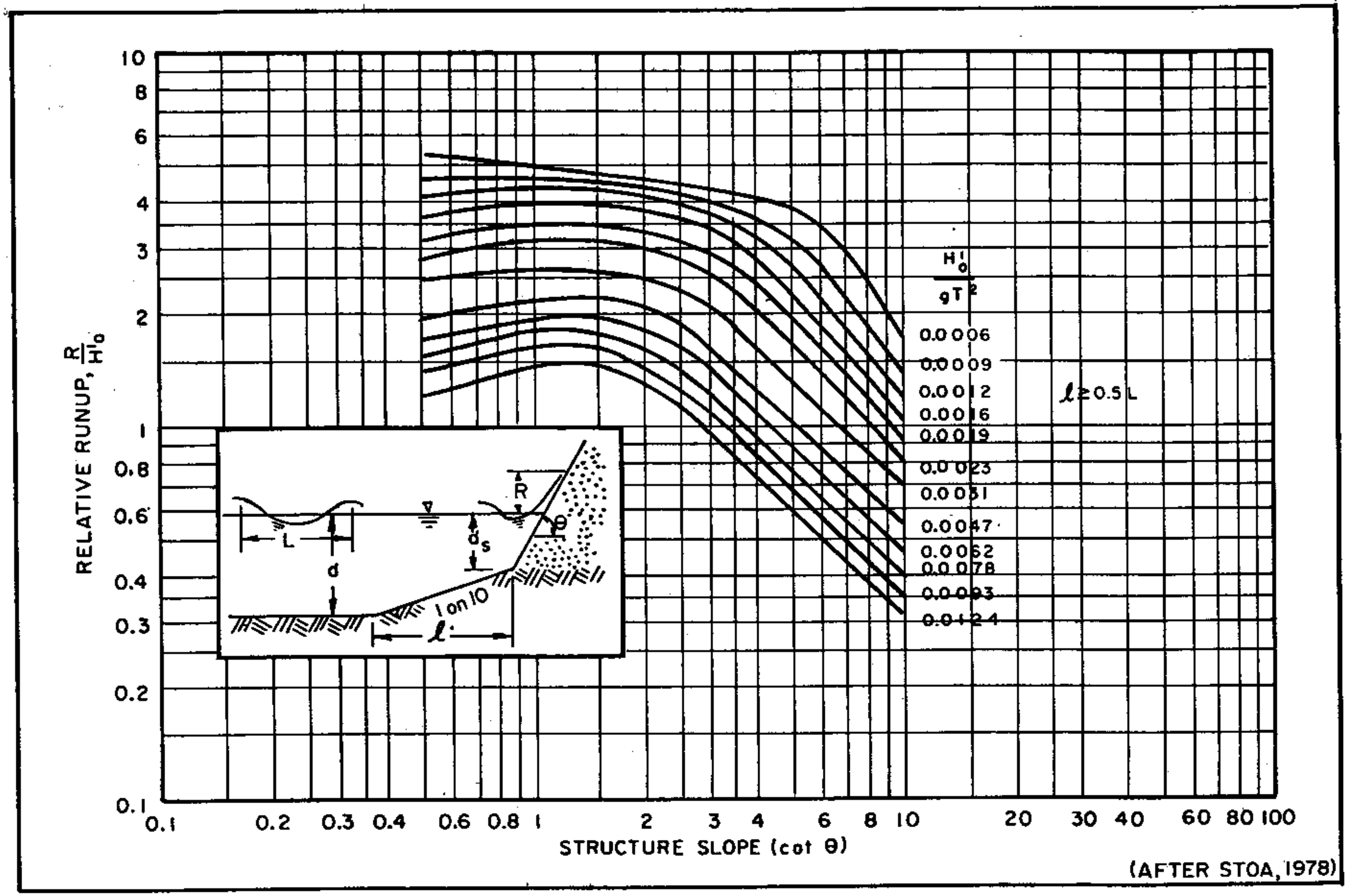


FIGURE 73

Relative Runup, R/H'_o, for a Smooth Embankment or Revetment Fronted by a 1-on-10 Bottom Slope for Relative Depth, $d_s/H'_o = 0.6$

Revetment Fronted by a 1-on-10 Bottom Slope for Relative Depth, dÚs¿/H'Úo¿ = 0.6]

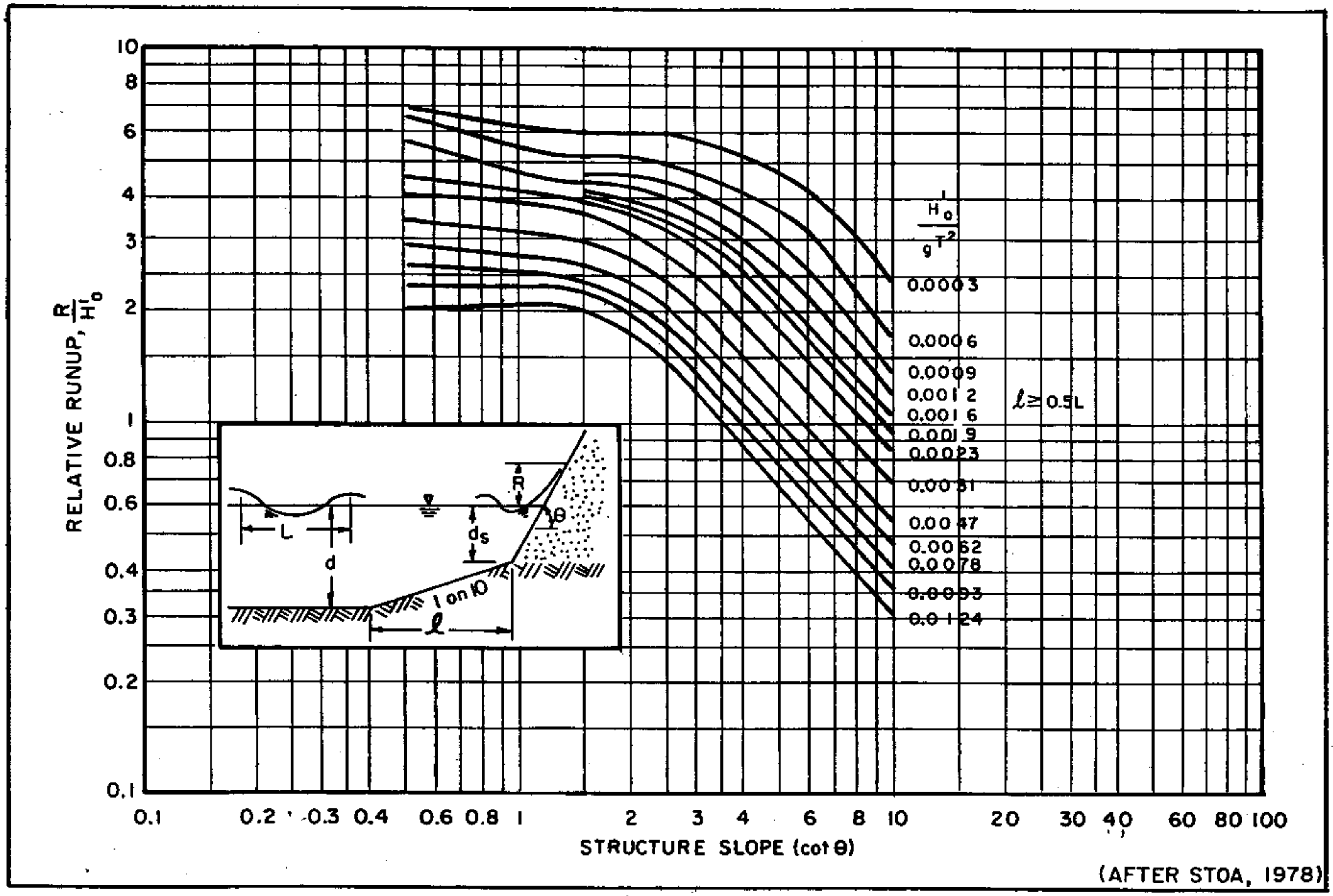


FIGURE 74

Relative Runup, R/H_o', for a Smooth Embankment or Revetment Fronted by a 1-on-10 Bottom Slope for Relative Depth, $d_s/H_o' = 1.0$

Revetment Fronted by a 1-on-10 Bottom Slope for Relative Depth, dÚs¿/H'Úo¿ = 1.0]

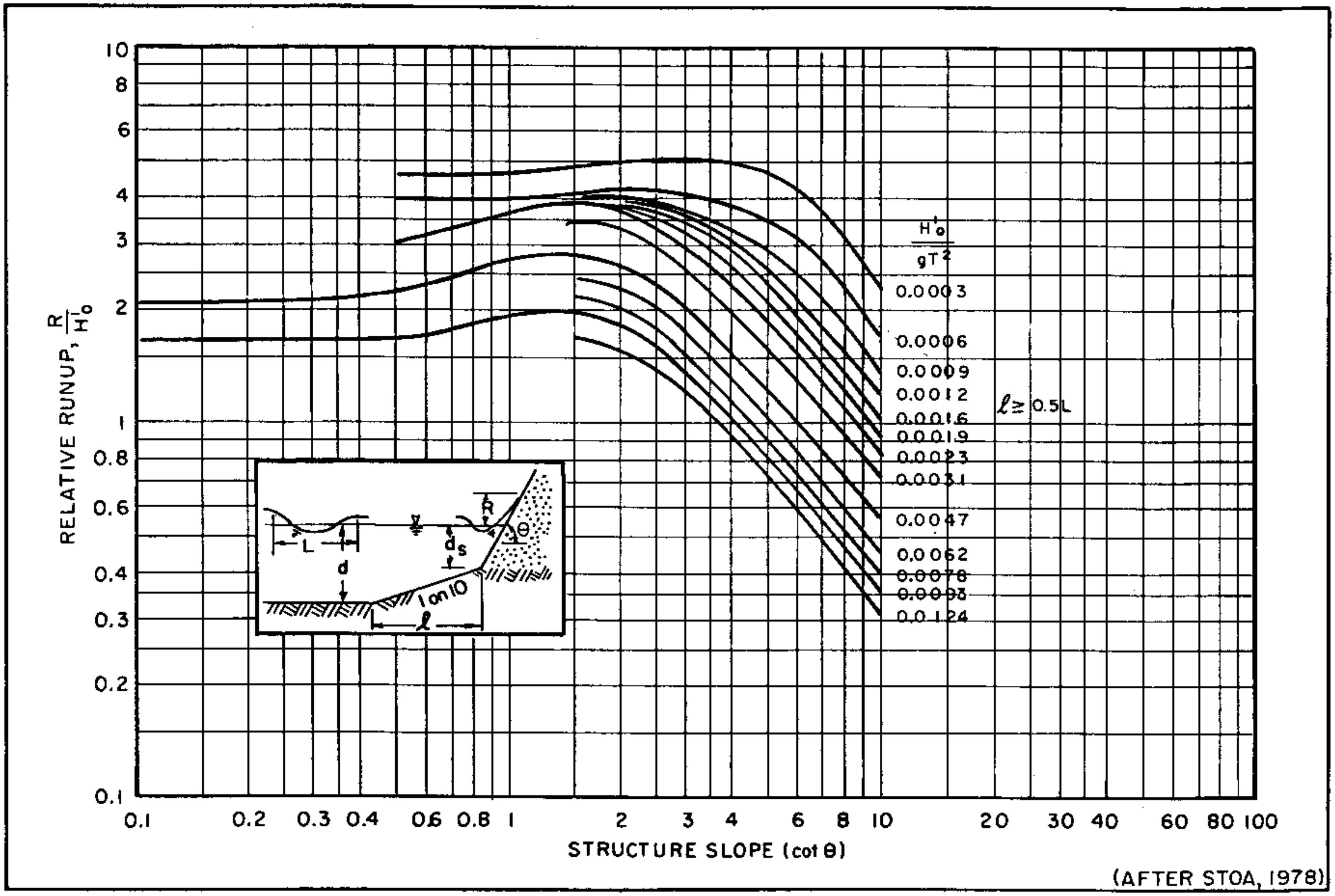


FIGURE 75

Relative Runup, R/H'_o, for a Smooth Embankment or Revetment Fronted by a 1-on-10 Bottom Slope for Relative Depth, $d_s/H'_o = 1.5$

Revetment Fronted by a 1-on-10 Bottom Slope for Relative Depth, dÚs¿/H'Úo¿ = 1.5]

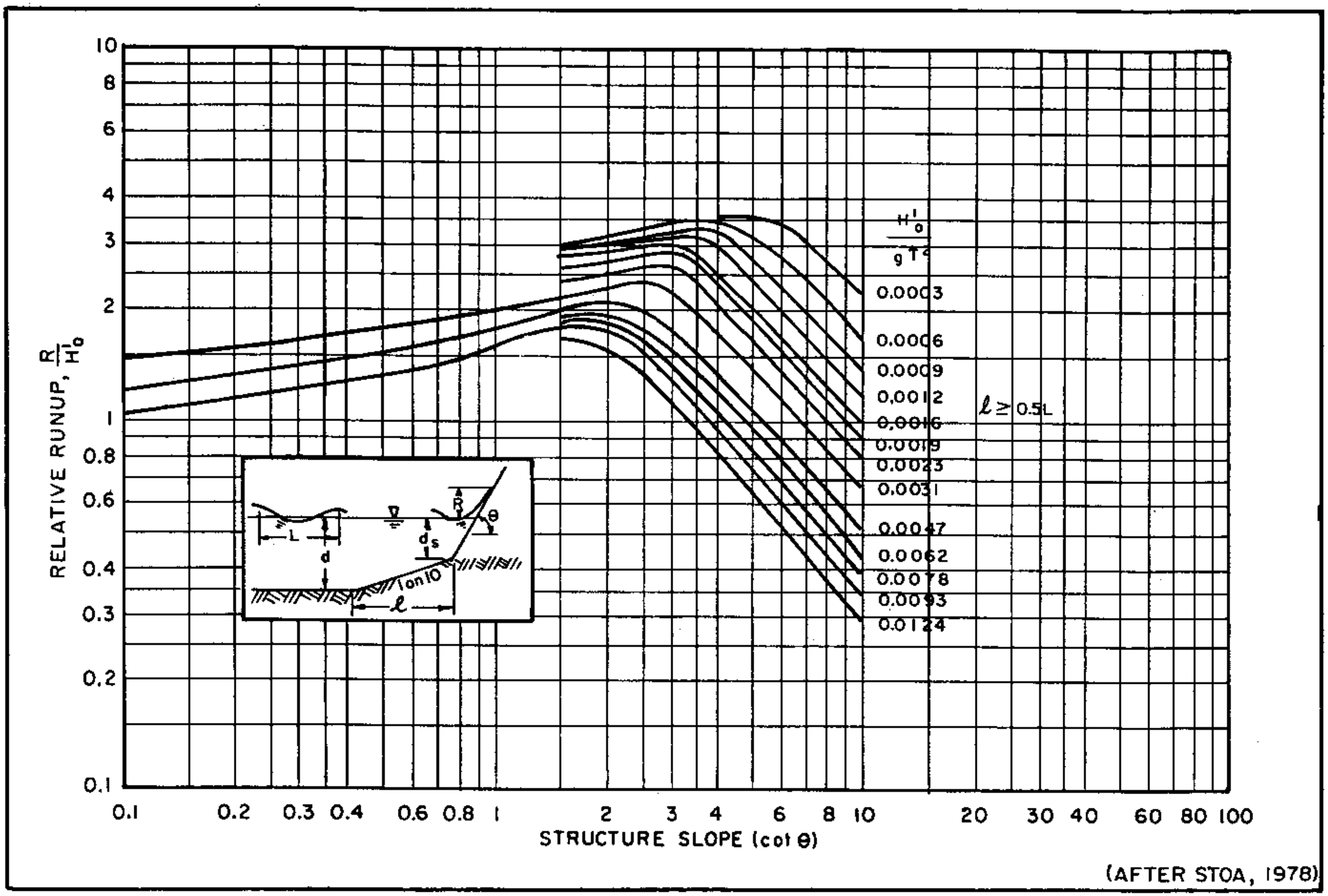


FIGURE 76
Relative Runup, R/H_o', for a Smooth Embankment or Revetment Fronted by a 1-on-10 Bottom Slope for Relative Depth, $d_s/H_o' = 3.0$

Revetment Fronted by a 1-on-10 Bottom Slope for Relative Depth, dÚs¿/H'Úo¿ = 3.0]

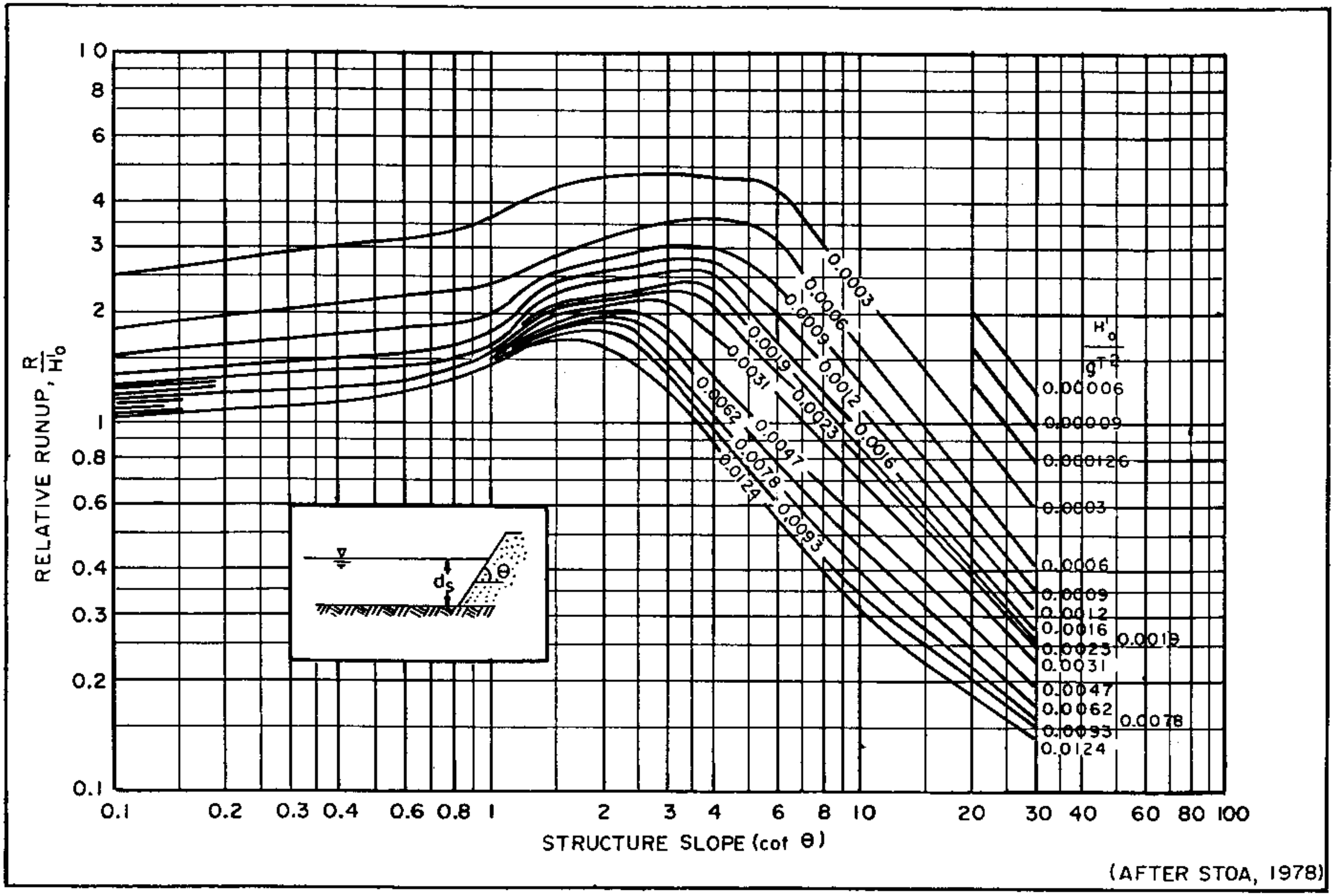


FIGURE 77

Relative Runup, R/H'_o, for a Smooth Embankment or Revetment for Relative Depth, $d_s/H'_o = 5.0$

Revetment for Relative Depth, dÚs¿/H'Úo¿ = 5.0]

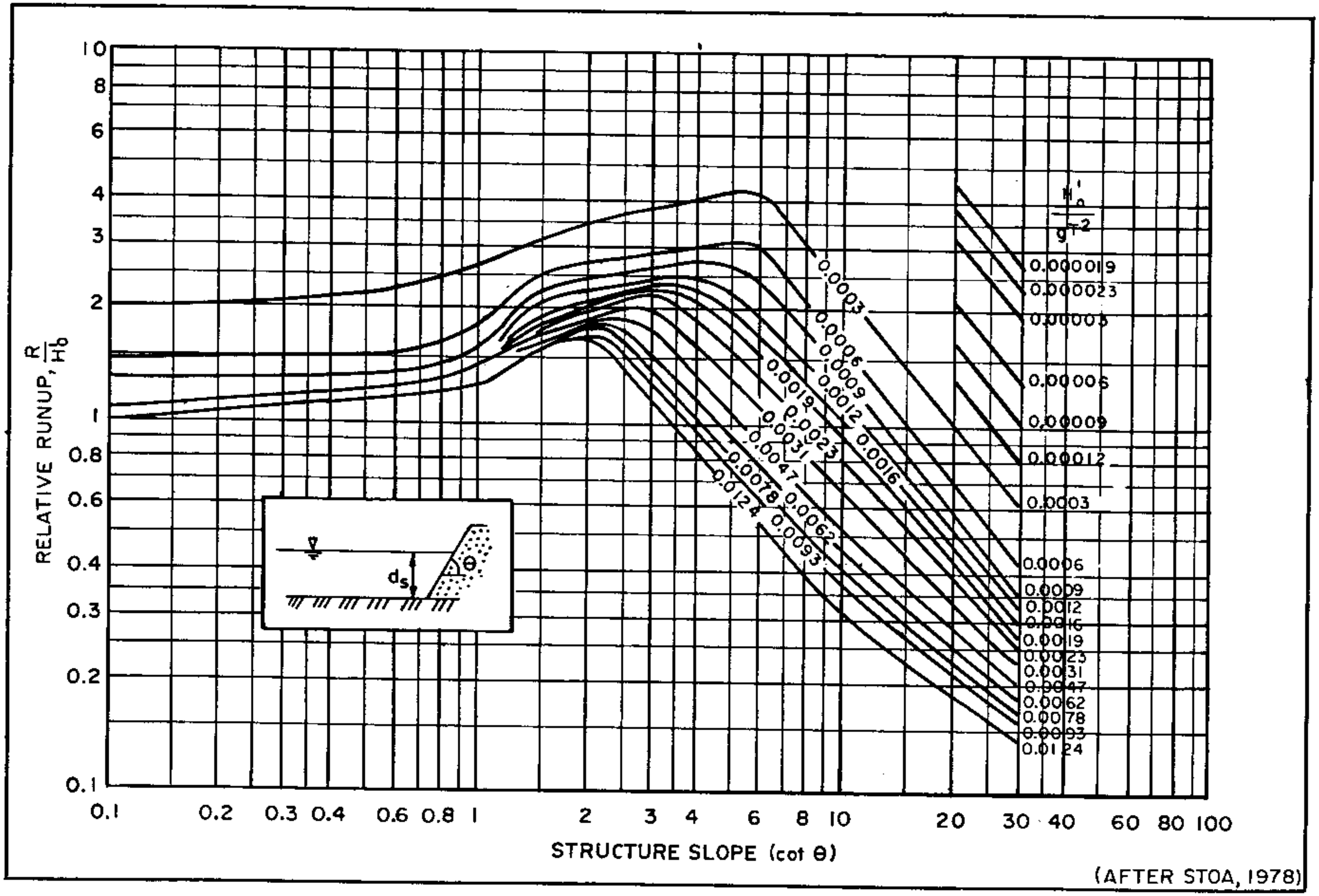


FIGURE 78
Relative Runup, R/H'_o, for a Smooth Embankment or Revetment for Relative Depth, $d_s/H'_o = 8.0$

Revetment for Relative Depth, dÚs¿/H'Úo¿ = 8.0]

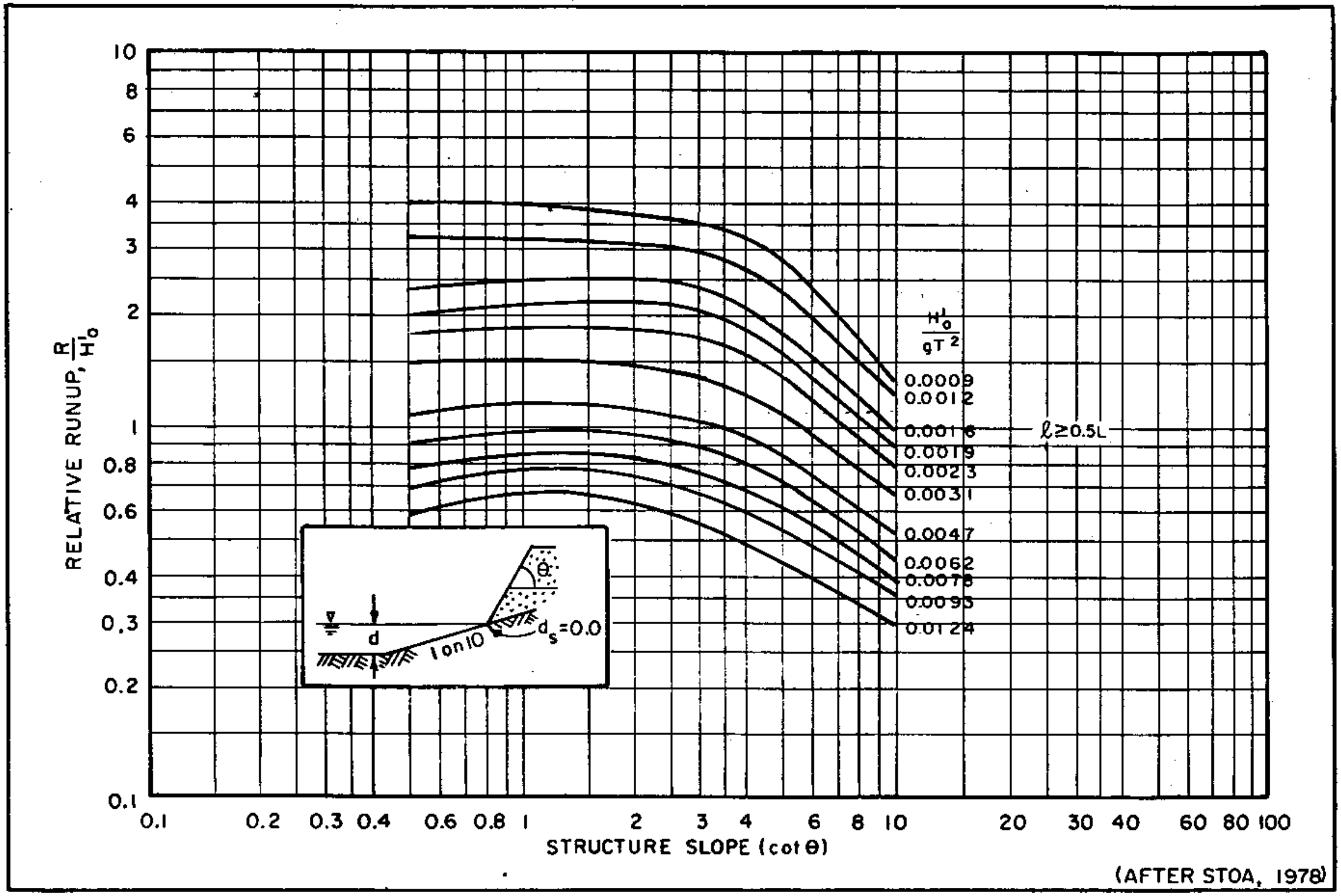


FIGURE 79

Relative Runup, R/H'_o, for a Smooth Embankment or Revetment With Water Depth at Toe, $d_s = 0.0$, and Relative Depth at Toe of 1-on-10 Bottom Slope, $d/H'_o = 3.0$

Revetment With Water Depth at Toe, dÚs¿ = 0.0, and
Relative Depth at Toe of 1-on-10 Bottom Slope, d/H'Úo¿ =
3.0]

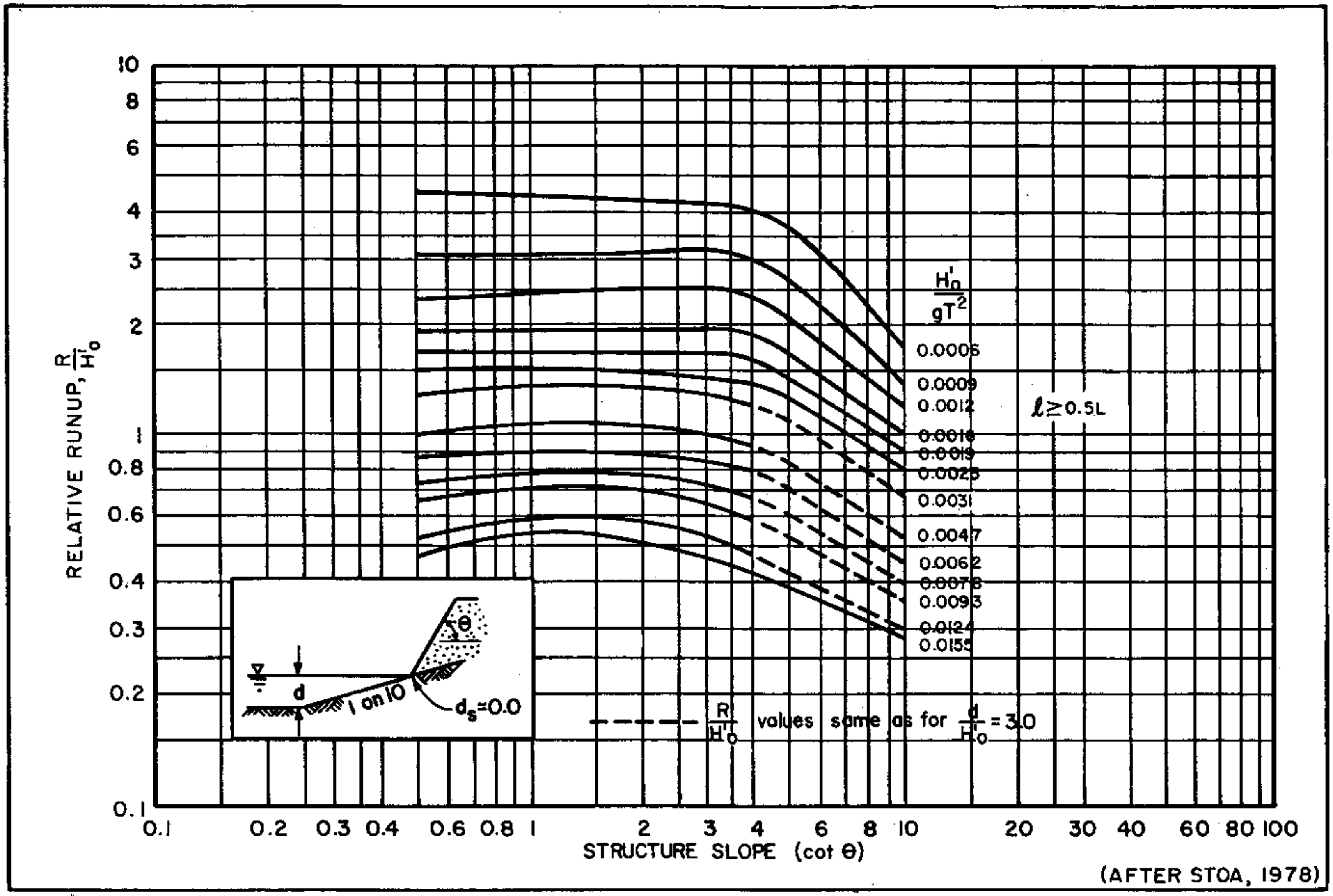


FIGURE 80

Relative Runup, R/H_o', for a Smooth Embankment or Revetment With Water Depth at Toe, $d_s = 0.0$, and Relative Depth at Toe of 1-on-10 Bottom Slope, $d/H_o' = 5.0$

Revetment With Water Depth at Toe, dÚs¿ = 0.0, and
Relative Depth at Toe of 1-on-10 Bottom Slope, d/H'Úo¿ =
5.0]

26.2-112

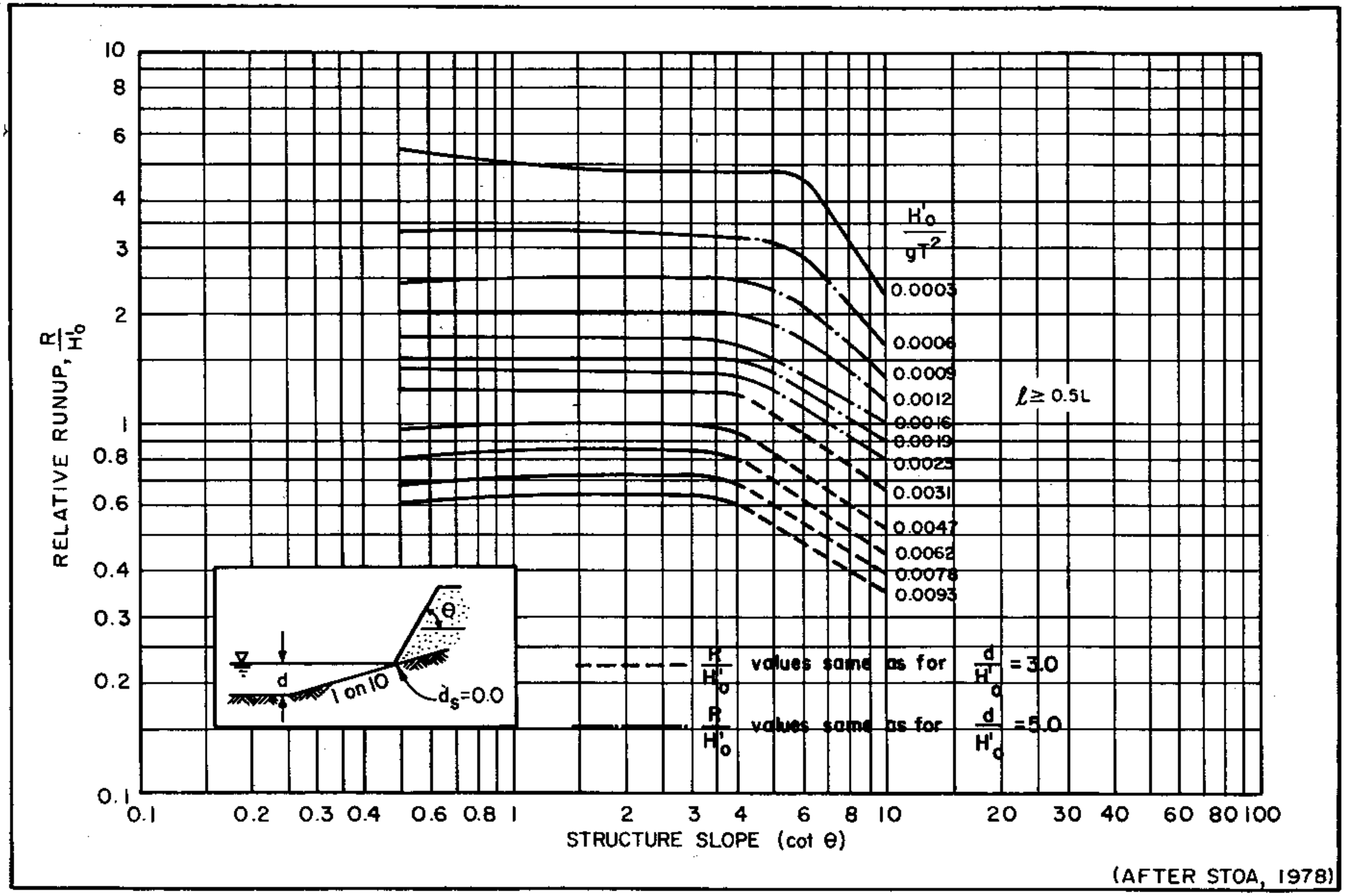


FIGURE 81

Relative Runup, R/H'_o, for a Smooth Embankment or Revetment With Water Depth at Toe, $d_s = 0.0$, and Relative Depth at Toe of 1-on-10 Bottom Slope, $d/H'_o = 8.0$

Revetment With Water Depth at Toe, dÚs¿ = 0.0, and
Relative Depth at Toe of 1-on-10 Bottom Slope, d/H'Úo¿ =
8.0]

EXAMPLE PROBLEM 13 (Continued)

(3) Find H'_o/gT^2:

$$\frac{H'_o}{gT^2} = \frac{3}{(32.2)(3)^2} = 0.0104$$

(4) From Figure 72 for cot [theta] = 1.5 and H'_o/gT^2 = 0.0104:

$$\frac{R}{H'_o} = 1.80$$

(5) Using Equation (3-3), find R:

$$R = (H'_o)(R/H'_o)(0.60)$$

$$R = (3)(1.80)(0.60) = 3.24 \text{ feet}$$

$$R = 3.2 \text{ feet}$$

EXAMPLE PROBLEM 14

Given:
a. The equivalent unrefracted deepwater wave height, H'_o = 6 feet
b. Water depth at structure toe, d_s = 15 feet
c. Wave period, T = 5 seconds
d. Structure slope, cot [theta] = 1.5
e. Slope in front of structure, m = 1:10 for slope length, *l = 100 feet

Find: Runup for a quarrystone revetment.

Solution: (1) Find L :

$$L_o = (g/2[pi])T^2 = (32.2/2[pi])(5)^2 = 128 \text{ feet}$$

(2) Determine depth at toe of 1:10 slope:

$$d = d_s + {*l}_m$$

$$d = 15 + (100)\left(\frac{1}{10}\right) = 25 \text{ feet}$$

(3) Find d/L_o:

$$\frac{d}{L_o} = \frac{25}{128} = 0.195$$

(4) From Figure 2 for d/L_o = 0.195:

$$\frac{d}{} = 0.22$$

EXAMPLE PROBLEM 14 (Continued)

d 25
L = ÄÄÄÄ = ÄÄÄÄÄÄ = 114 feet
0.22 0.22

(5) Determine if 0. 5 L:

0.5 L = 57 feet

*I = 100 feet

THEREFORE: *I > / = 0.5 L

(6) Determine dÚs¿/H'Úo¿:

dÚs¿ 15
ÄÄÄÄ = ÄÄ = 2.5
H'Úo¿ 6

dÚs¿/H'Úo¿ = 2.5 < 3 and *I > / = 5 L; therefore, use Figures 73 through 76

(7) Find H'Úo¿/g TÀ2Ù:

H'Úo¿ 6
ÄÄÄÄÄÄÄ = ÄÄÄÄÄÄÄÄÄÄÄÄÄ = 0.0075
g TÀ2Ù (32.2) (5)À2Ù

(8) Interpolate between Figures 75 and 76 to find R/H'Úo¿ for H'Úo¿/g TÀ2Ù = 0.0075: ø

R
ÄÄÄÄÄ = 2. 03
H'Úo¿

(9) Using Equation (3-3), find R:

R = (H'Úo¿) (R/H'Úo¿) (0.60)

R = (6) (2.03) (0.60) = 7.31 feet

R = 7.3 feet

(3) Case 2: Embankment or Revetment, Concrete Armor. Runup on revetments protected by concrete armor units is determined from the appropriate smooth-slope curvEs, Figures 72 through 81, for the appropriate dÚs¿/H'Úo¿, cot [theta], and H'Úo¿/g TÀ2Ù. The rough-slope runup correction factor, r, is found in Table 6 for special concrete shapes. The runup scale-effect correction factor, k, is equal to 1.03.

R = (H'Úo¿) (R/H'Úo¿) (r) (k)

(3-4)

R = (H'Úo¿) (R/H'Úo¿) (r) (1.03)

WHERE: r is found in Table 6

TABLE 6

Rough-Slope Runup Correction Factor, r, for Concrete Armor Units

Unit	Number of Layers	Placement	r	Structure Slope (cot [theta])
Dolos	2	Random	0.45	1.3 to 3.0
Modified cube . . .	2	Random	0.48	1.3 to 3.0
	1	Uniform	0.62	1.5
	1	Uniform	0.73	2.0
	1	Uniform	0.55	3.0
Quadripod	2	Random and uniform	0.51	1.3 to 3.0
Tetrapod.	2	Random	0.45	1.3 to 3.0
	2	Uniform	0.51	1.3 to 3.0
Tribar.	2	Random	0.45	1.3 to 3.0
	1	Uniform	0.50	1.3 to 3.0
Gobi blocks	1	Uniform	0.93	1.3 to 3.0
Stepped slopes. . .	N.A.	Vertical risers	0.75	1.3 to 3.0
		Curved risers	0.86	1.3 to 3.0

(STOA, 1979)

EXAMPLE PROBLEM 15

Given:
a. The equivalent unrefracted deepwater wave height, H'_o = 10 feet
b. Water depth at structure toe, d_s = 30 feet
c. Wave period, T = 8 seconds
d. Structure slope, cot [theta] = 1.5

Find: Runup for a revetment armored with two layers of randomly placed tetrapods.

Solution: (1) Find d_s/H'_o:

$$\frac{d_s}{H'_o} = \frac{30}{10} = 3\text{: therefore, use Figure 72}$$

(2) Find $H'_o/g T^2$:

$$\frac{H'_o}{g T^2} = \frac{10}{(32.2)\ (8)^2} = 0.0049$$

EXAMPLE PROBLEM 15 (Continued)

(3) From Figure 72 for cot [theta] = 1.5 and H'_o/gT^2 = 0.0049:

$$\frac{R}{H'_o} = 1.90$$

(4) From Table 6 for tetrapod armor units, randomly placed: r = 0.45

(5) Using Equation (3-4), find R:

$R = (H'_o)(R/H'_o)(r)(1.03$

$R = (10)(1.9)(0.45)(1.03) = 8.81$ feet

$R = 8.8$ feet

Compared to 12.6 feet for the quarrystone revetment in Example Problem 2, the tetrapod armor units reduce the runup by 30 percent.

(4) Cases 3, 4, and 5: Breakwater, Rubble-Mound. Wave runup on a rubble-mound breakwater is a function of the height of the impermeable core above the bottom, as well as of the parameters affecting runup on an embankment. The first step is to determine if the structure has a low, medium, or high core. The classification of core height is given in Table 7. Also to be found in Table 7 is the subsection that applies to each core height. The parameter, h_c, is the height of the core above bottom.

TABLE 7

Classification of Relative Core Height, h_c/d_s

Classification	Relative Core Height	Case	Subsection
Low	$h_c/d_s \leq 0.75$	3	3.4.b.(4)(a)
Medium.	$0.75 < h_c/d_s < 1.1$	4	3.4.b.(4)(b)
High.	$h_c/d_s \geq 1.1$	5	3.4.b.(4)(c)

(a) Case 3: low core height: $h_c/d_s \leq 0.75$. If $d_s/H'_o \geq 3$ and $1.25 \leq \cot [\theta] \leq 5$, find R/H'_o from Figure 82, 83, or 84. The rough-slope runup correction factor, r, is 1.00, and the runup scale-effect correction factor, k, is 1.06.

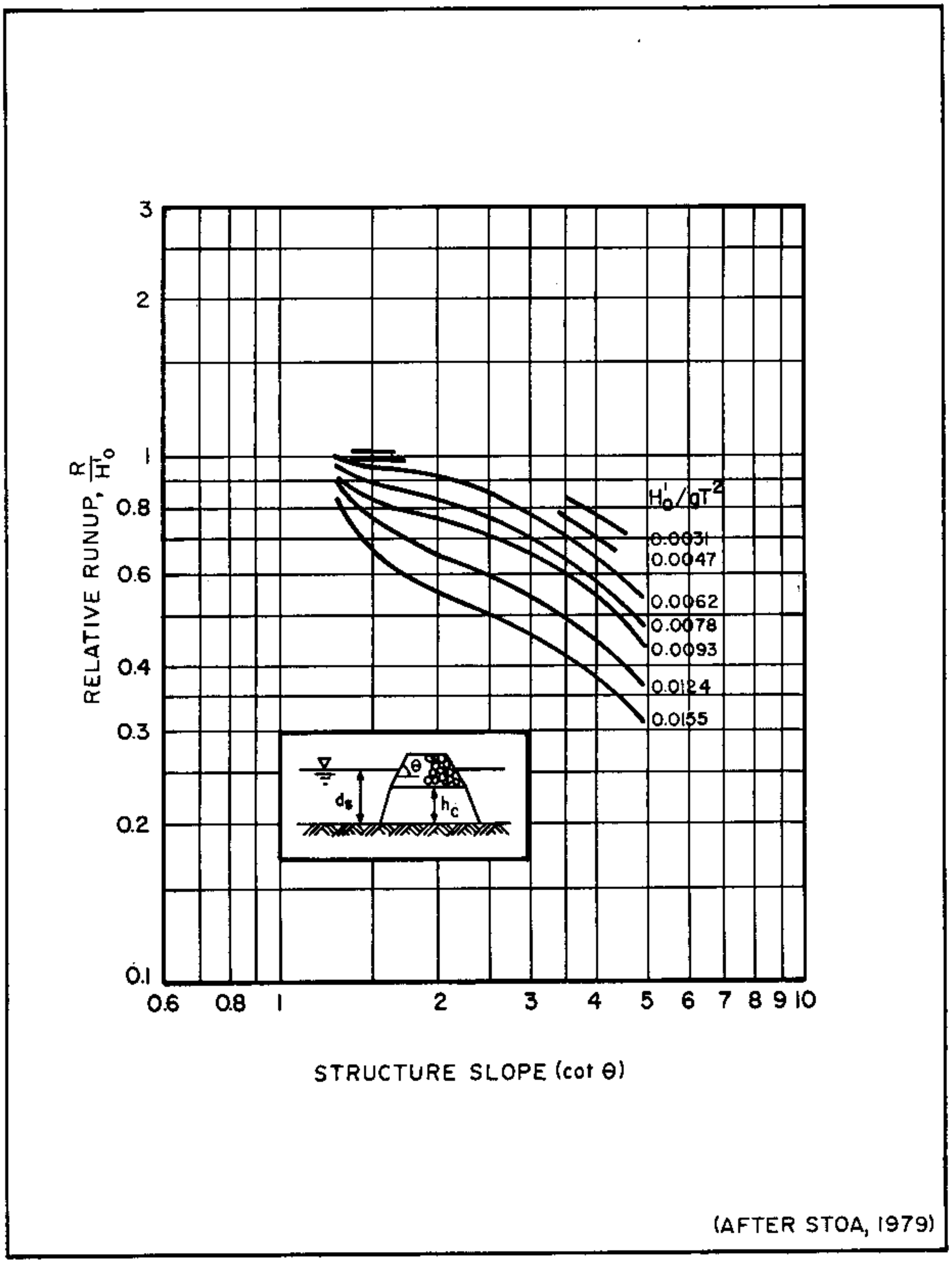


FIGURE 82
Relative Runup, R/H_o', for a Rubble-Mound Breakwater for Relative Depth, $d_s/H_o' = 3.0$

for Relative Depth, dÚs¿/H'Úo¿ = 3.0

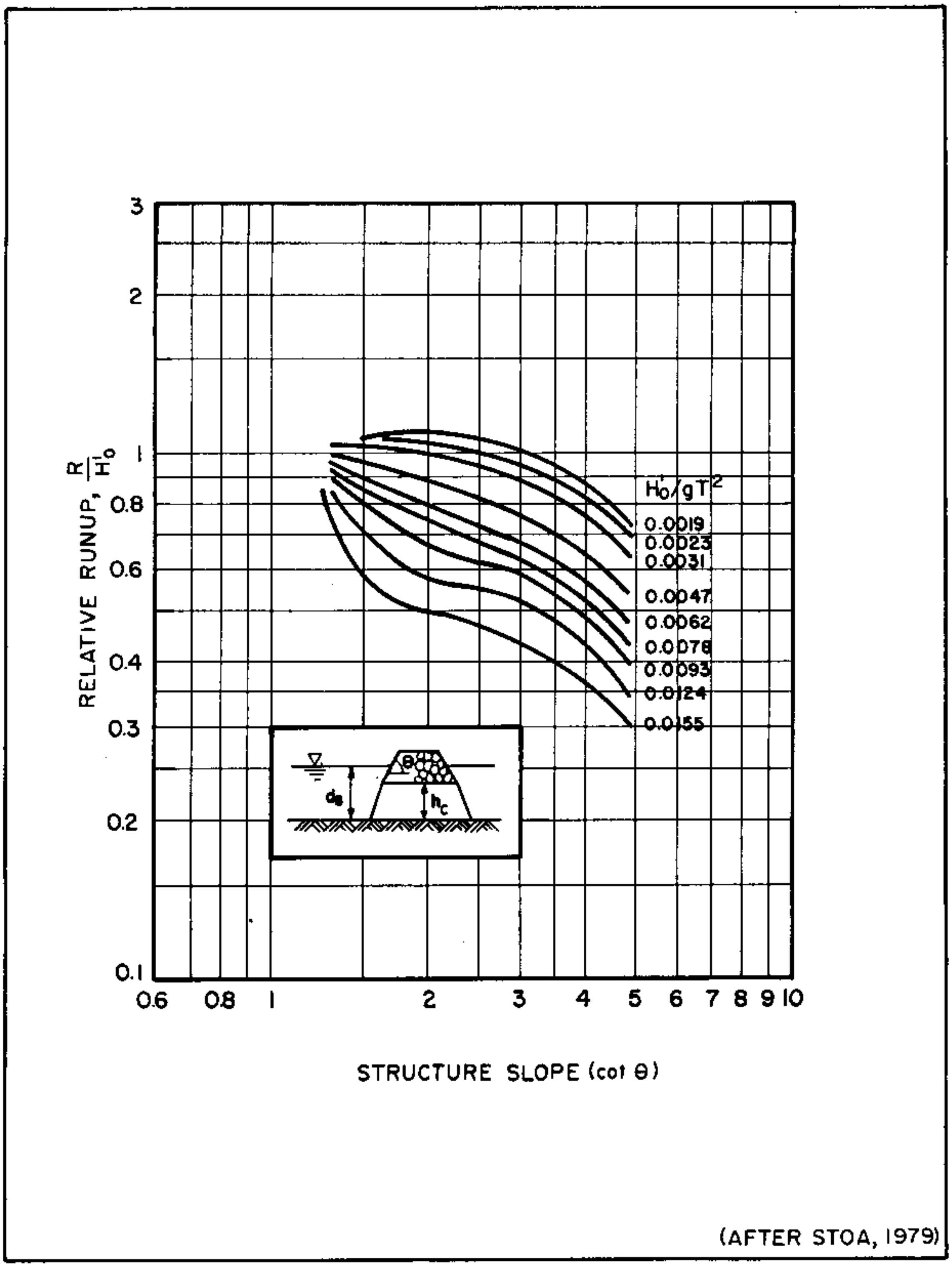


FIGURE 83
Relative Runup, R/H_o', for a Rubble-Mound Breakwater for Relative Depth, $d_s/H_o' = 5.0$

for Relative Depth, dÚs¿/H'Úo¿ = 5.0

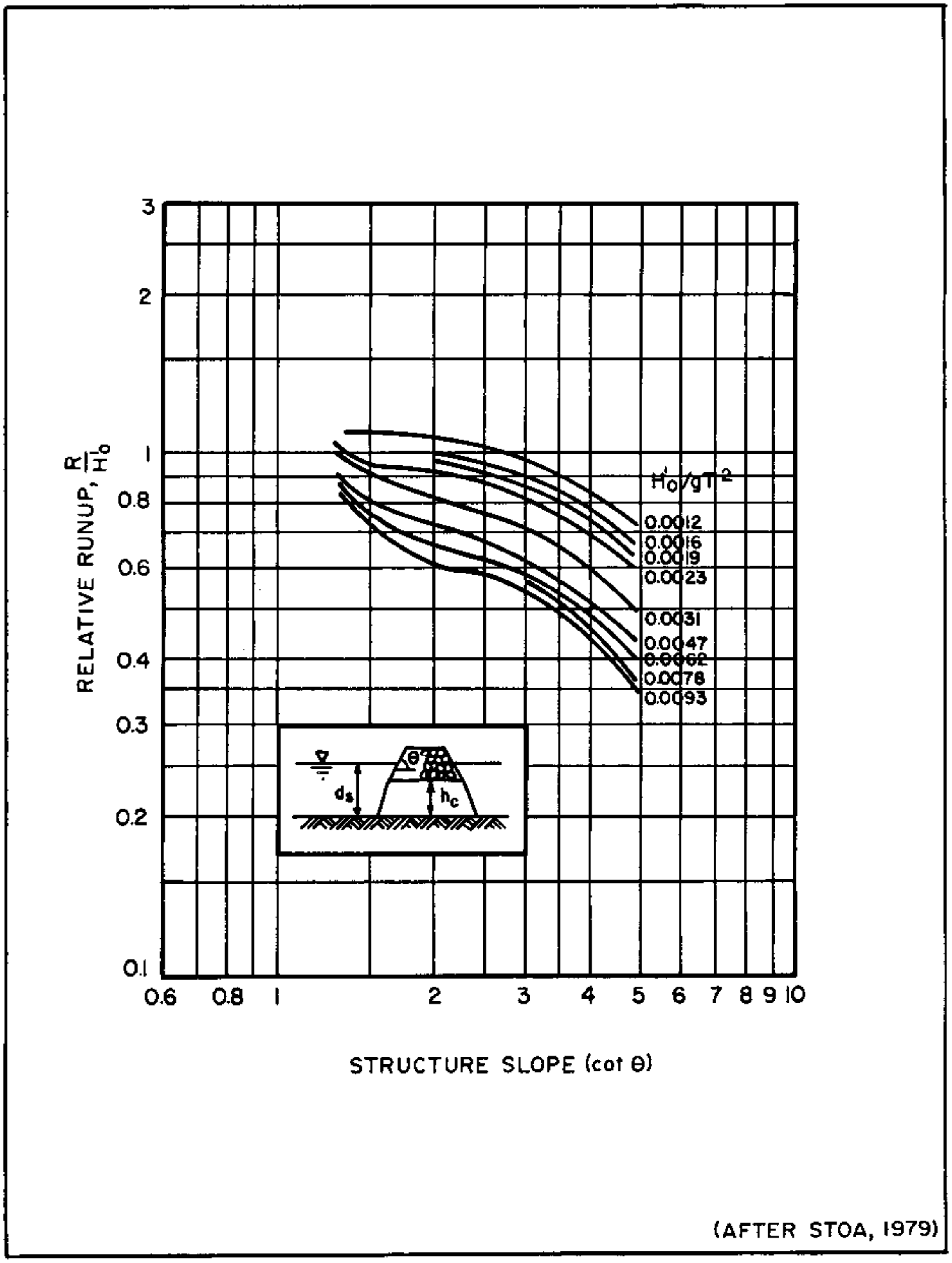


FIGURE 84
Relative Runup, R/H'_o, for a Rubble-Mound Breakwater for Relative Depth, $d_s/H'_o = 8.0$

for Relative Depth, dÚs¿/H'Úo¿ = 8.0

$$R = (H'_o)(R/H'_o)(r)(k)$$

$$R = (H'_o)(R/H'_o)(1.06) \qquad (3\text{-}5)$$

If $d_s/H'_o < 3$ and/or $1.25 > \cot\theta > 5$, find the rough-slope runup correction factor, r, from Table 8. The runup scale-effect correction factor, k, is 1.06. Then the runup is determined for the appropriate smooth-slope curve chosen from Figures 72 through 75. If $1.5 < d_s/H'_o < 3$, interpolate between Figures 75 and 76 in order to determine runup.

$$R = (H'_o)(R/H'_o)(r)(k)$$

$$R = (H'_o)(R/H'_o)(r)(1.06) \qquad (3\text{-}6)$$

WHERE: r is found in Table 8

TABLE 8
Rough-Slope Runup Correction Factor, r,
for a Rubble-Mound Breakwater

Structure Slope (cot [theta])	r
1.25	0.57
1.50	0.45
2.00	0.44
2.50	0.42
3.00	0.44
4.00	0.48
5.00	0.48

(STOA, 1979)

EXAMPLE PROBLEM 16

Given:
- a. The equivalent unrefracted deepwater wave height, H'_o = 8 feet
- b. Water depth at structure toe, d_s = 40 feet
- c. Wave period, T = 5 seconds
- d. Structure slope, cot [theta] = 1.5
- e. Height of core, h_c = 28 feet

Find: Runup for a quarrystone breakwater.

Solution: (1) Determine relative core height, h_c/d_s:

$$\frac{h_c}{d_s} = \frac{28}{40} = 0.7$$

From Table 7: relative core height is low.

EXAMPLE PROBLEM 16 (Continued)

(2) Find dÚs¿/H'Úo¿:

```
 dÚs¿   40
ÄÄÄÄÄ = ÄÄ = 5
H'Úo¿    8
```

dÚs¿/HÚo¿ > / = 3 and 1.25 < / = cot [theta] < / = 5; therefore, use Figure 82, 83, or 84

(3) Find HÚo¿/g TÀ2Ù:

```
H'Úo¿           8
ÄÄÄÄÄÄ = ÄÄÄÄÄÄÄÄÄÄÄÄ = 0.0099
g TÀ2Ù   (32.2)(5)À2Ù
```

(4) From Figure 83 for cot [theta] = 1.5 and HÚo¿/g TÀ2Ù = 0.0099:

```
  R
ÄÄÄÄÄ = 0.78
H'Úo¿
```

(5) Using Equation (3-5), find R:

R = (H'Úo¿)(R/H'Úo¿)(1.06)

R = (8) (0.78) (1.06) = 6.61 feet

R = 6.6 feet

EXAMPLE PROBLEM 17

Given:
- a. Equivalent unrefracted deepwater wave height, H'Úo¿ = 8 feet
- b. Water depth at structure toe, dÚs¿ = 20 feet
- c. Wave period, T = 5 seconds
- d. Structure slope, cot [theta] = 2.0
- e. Height of core, h = 15 feet
- f. Structure situated on flat bottom.

Find: Runup for a quarrystone breakwater.

Solution: (1) Determine relative core height, hÚc¿/dÚs¿:

```
hÚc¿   15
ÄÄÄÄ = ÄÄ = 0.75
dÚs¿   20
```

From Table 7: relative core height is low

EXAMPLE PROBLEM 17 (Continued)

(2) Find d_s/H'_o:

$$\frac{d_s}{H'_o} = \frac{20}{8} = 2.5$$

$d_s/H'_o < 3$ and $1.25 >$ cot [theta] > 5; therefore, choose figure to use from Figures 72 through 75.

Since structure is on flat bottom, use Figure 72.

(3) Find $H'_o/g\,T^2$:

$$\frac{H'_o}{g\,T^2} = \frac{8}{(32.2)(5)^2}\ 0.0099$$

(4) From Figure 72, for cot [theta] = 2.0 and $H_o/g\,T^2$= 0.0099:

$$\frac{R}{H'_o} = 1.85$$

(5) From Table 8 for cot [theta] = 2.0: r = 0.44

(6) Using Equation (3-6), find R:

$R = (H'_o)(R/H'_o)(r)(1.06)$

$R = (8)(1.85)(0.44)(1.06) = 6.90$ feet

R = 6.9 feet

(b) Case 4: medium core height: $0.75 < h_c/d_s < 1.1$. Find R/H'_o from Figures 72 through 78 for the appropriate d_s/H'_o and bottom configuration. The rough-slope runup correction factor, r, is 0.52 and the runup scale-effect correction factor, k, is 1.06.

$$R = (H'_o)(R/H'_o)(r)(k) \qquad (3\text{-}7)$$

$$R = (H'_o)(R/H'_o)(0.52)(1.06)$$

EXAMPLE PROBLEM 18

Given:
a. The equivalent unrefracted deepwater wave height, H'_o = 10 feet
b. Water depth at structure toe, d = 50 feet
c. Wave period, T = 8 seconds
d. Structure slope, cot [theta] = 2.5
e. Height of core, h = 43 feet

EXAMPLE PROBLEM 18 (Continued)

Find: Runup for a quarrystone breakwater.

Solution: (1) Determine relative core height, h_c/d_s:

$$\frac{h_c}{d_s} = \frac{43}{50} = 0.86$$

From Table 7: relative core height is medium

(2) Find d_s/H'_o:

$$\frac{d_s}{H'_o} = \frac{50}{10} = 5$$

$d_s/H'_o = 5$; therefore, use Figure 77

(3) Find $H'_o/g\,T^2$:

$$\frac{H'_o}{g\,T^2} = \frac{10}{(32.2)(8)\,T^2} = 0.0049$$

(4) From Figure 77 for cot [theta] = 2.5 and $H'_o/g\,T^2$ = 0.0049:

$$\frac{R}{H'_o} = 2.00$$

(5) Using Equation (3-7), find R:

$R = (H'_o)\ (R/H'_o)\ (0.52)\ (1.06)$

$R = (10)\ (2.0)\ (0.52)\ (1.06) = 11.0$ feet

R = 11.0 feet

(c) Case 5: high core height. Find R/H'_o from Figures 72 through 78 for the appropriate d_s/H'_o and bottom configuration. The rough-slope runup correction factor, r, is 0.60, and the runup scale-effect correction factor, k, is 1.00.

$$R = (H'_o)\ (R/H'_o)\ (r)\ (k) \qquad (3\text{-}8)$$

$$R = (H'_o)\ (R/H'_o)\ (0.60)$$

EXAMPLE PROBLEM 19

Given:
a. The equivalent unrefracted deepwater wave height, H'_o = 20 feet
b. Water depth at structure toe, d_s = 60 feet
c. Wave period, T = 12 seconds
d. Structure slope, cot [theta] = 1.5
e. Height of core, h_c = 72 feet

Find: Runup for a quarrystone breakwater.

Solution: (1) Determine relative core height, h_c/d_s :

$$\frac{h_c}{d_s} = \frac{72}{60} = 1.2$$

From Table 7: relative core height is high

(2) Find d_s/H'_o:

$$\frac{d_s}{H'_o} = \frac{60}{20} = 3.0$$

d_s/H'_o = 3.0; therefore, use Figure 72

(3) Find $H'_o /g T^2$:

$$\frac{H'_o}{g T^2} = \frac{20}{(32.2)\ (12)^2} = 0.0043$$

(4) From Figure 72 for cot [theta] = 1.5 and $H'_o/g T^2$ = 0.0043:

$$\frac{R}{H'_o} = 2.05$$

(5) Using Equation (3-8), find R:

$$R = (H'_o)\ (R/H'_o)\ (0.6)$$

$$R = (20)\ (2.05)\ (0.60) = 24.6 \text{ feet}$$

R = 24.6 feet

(5) Case 6: Breakwater, Concrete Armor. Find the relative runup, R/H'_o, on a smooth slope in Figures 72 through 78 for the appropriate d_s/H'_o and bottom configuration. Find the rough-slope runup correction factor, r, from Table 6. The runup scale-effect correction factor, k, is 1.03.

$$R = (H'_o)(R/H'_o)(r)(k)$$

$$R = (H'_o)(R/H'_o)(r)(1.03) \qquad (3-9)$$

WHERE: r is found in Table 6

EXAMPLE PROBLEM 20

Given: a. Equivalent unrefracted deepwater wave height, H'_o = 20 feet
b. Water depth at structure toe, d_s = 60 feet
c. Wave period, T = 12 seconds
d. Structure slope, cot [theta] = 1.5
e. Height of core, h_c = 72 feet

Find: Runup for a breakwater armored with one layer of uniformly placed tribars.

Solution: From Example Problem 19: R/H'_o = 2.05

From Table 6: r = 0.50

Using Equation (3-9), find R:

$R = (H'_o)(R/H'_o)(r)(1.03)$

R = (20)(2.05)(0.50)(1.03) = 21.1 feet

R = 21.1 feet

Note: Runup is reduced approximately 14 percent by using tribars instead of quarrystone.

(6) Case 7: Vertical Structures. Wave runup on a smooth-faced vertical structure located on a horizontal bottom for a nonbreaking or nonbroken wave is essentially equal to the incident wave height, H_i. Waves in shoaling water have nonlinear asymmetry in that the crest height is greater than the trough depression. This, in effect, raises the height of the mean water level at the wall by an amount, h_o, above the still water level. Runup, R, is calculated from the equation:

$$R = h_o + H_i \qquad (3-10)$$

WHERE: R = runup

h_o = amount by which the mean water level at the wall is raised above still water level

H_i = incident wave height at structure toe (as determined by the linear shoaling coefficient)

The value of h_o is determined by first finding $h_o/H = h_o/H_i$ from Figure 85. H_i is calculated from a given H'_o by first determining h_o from $H'_o = H_o K_R$ (linear shoaling) for a given K_R and then using Figure 2 to find H/H'_o for the calculated value of d_s/L_o.

This procedure is not applicable if the wave has broken.

EXAMPLE PROBLEM 21

Given:
a. Equivalent unrefracted deepwater wave height, H'_o = 5 feet
b. Water depth at structure toe, d_s= 15 feet
c. Wave period, T = 4 seconds
d. Refraction coefficient, K_R = 1.0

Find: The runup on a smooth-faced vertical wall.

Solution: (1) Find H_o:

$$H'_o = h_c K_R$$

$$H_o = \frac{H'_o}{K_R} = \frac{5}{1.0}$$

$$H_o = 5.0 \text{ feet}$$

(2) Find d_s/L_o:

$$L_o = (g/2 [pi])\, T^2 = (32.2/2 [pi])(4)^2 = 82.0 \text{ feet}$$

$$\frac{d_s}{L_o} = \frac{15}{82.0} = 0.183$$

(3) From Figure 2 for $d_s/L_o = 0.183$:

$$H/H'_o = 0.92$$

$$H = 0.92\, H'_o$$

$$H = (0.92)\ (5)$$

$$H = H_i = 4.6 \text{ feet}$$

(4) Find H/d_s; $H = H_i$:

$$\frac{H_i}{d_s} = \frac{4.6}{15} = 0.31$$

(5) Find H/T^2; $H = H_i$:

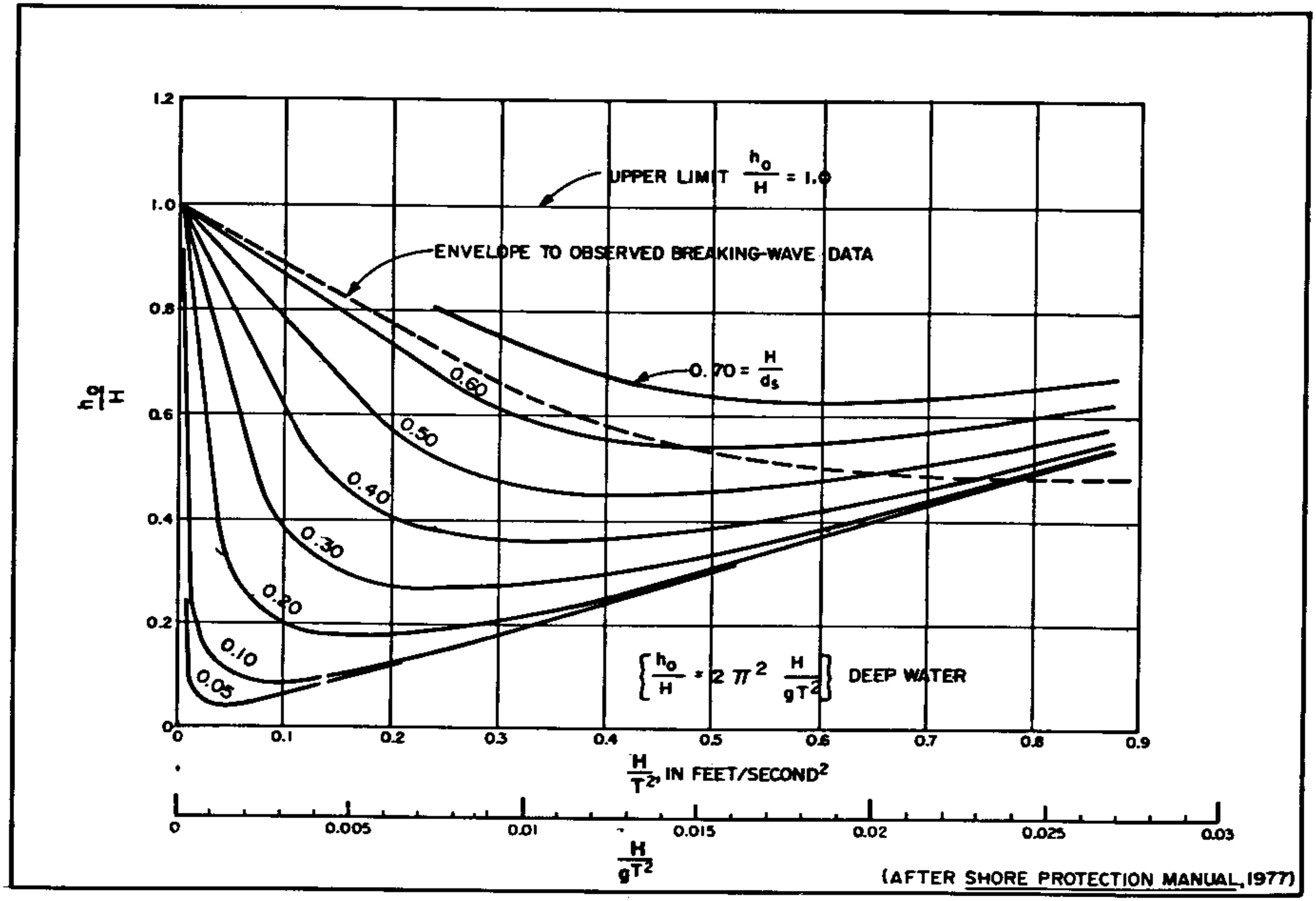


FIGURE 35
h_o/H Versus H/T^2 or $H/g\,T^2$

EXAMPLE PROBLEM 21 (Continued)

$$\frac{H_i}{T^2} = \frac{4.62}{(4)^2} = 0.29$$

(6) From Figure 85 for $H/d_s = 0.31$ and $H/T^2 = 0.29$; $H = H_i$:

$$\frac{h_o}{H_i} = 0.28$$

$$h_o = 0.28\ H_i$$

$$h_o = (0.28)\ (4.6) = 1.29$$

$$h_o = 1.3 \text{ feet}$$

(7) Using Equation (3-10), find R:

$$R = h_o + H_i$$

$$R = 1.3 + 4.6 = 5.9 \text{ feet}$$

$$R = 5.9 \text{ feet}$$

For shallow-water application, where the wave may break, relative wave runup, R/H'_o, for smooth-faced vertical structures and recurved seawalls fronted by nonhorizontal bottom slopes can be found in Figures 86, 87, and 88. The rough-slope runup correction factor, r, and the runup scale-effect correction factor, k, are both 1.00.

$$R = (H'_o)\ (R/H'_o)\ (r)(k)$$
$$R = (H'_o)\ (R/H'_o) \qquad (3\text{-}11)$$

EXAMPLE PROBLEM 22

Given: a. The equivalent unrefracted deepwater wave height, $H'_o = 3$ feet
b. Water depth at structure toe, $d_s = 4.5$ feet
c. Wave period, $T = 3$ seconds

Find: Runup on recurved Galveston-type seawall.

Solution: (1) Find d_s/H'_o:

$$\frac{d_s}{H'_o} = \frac{4.5}{3} = 1.5$$

(2) Find $H'_o\ /g\ T^2$:

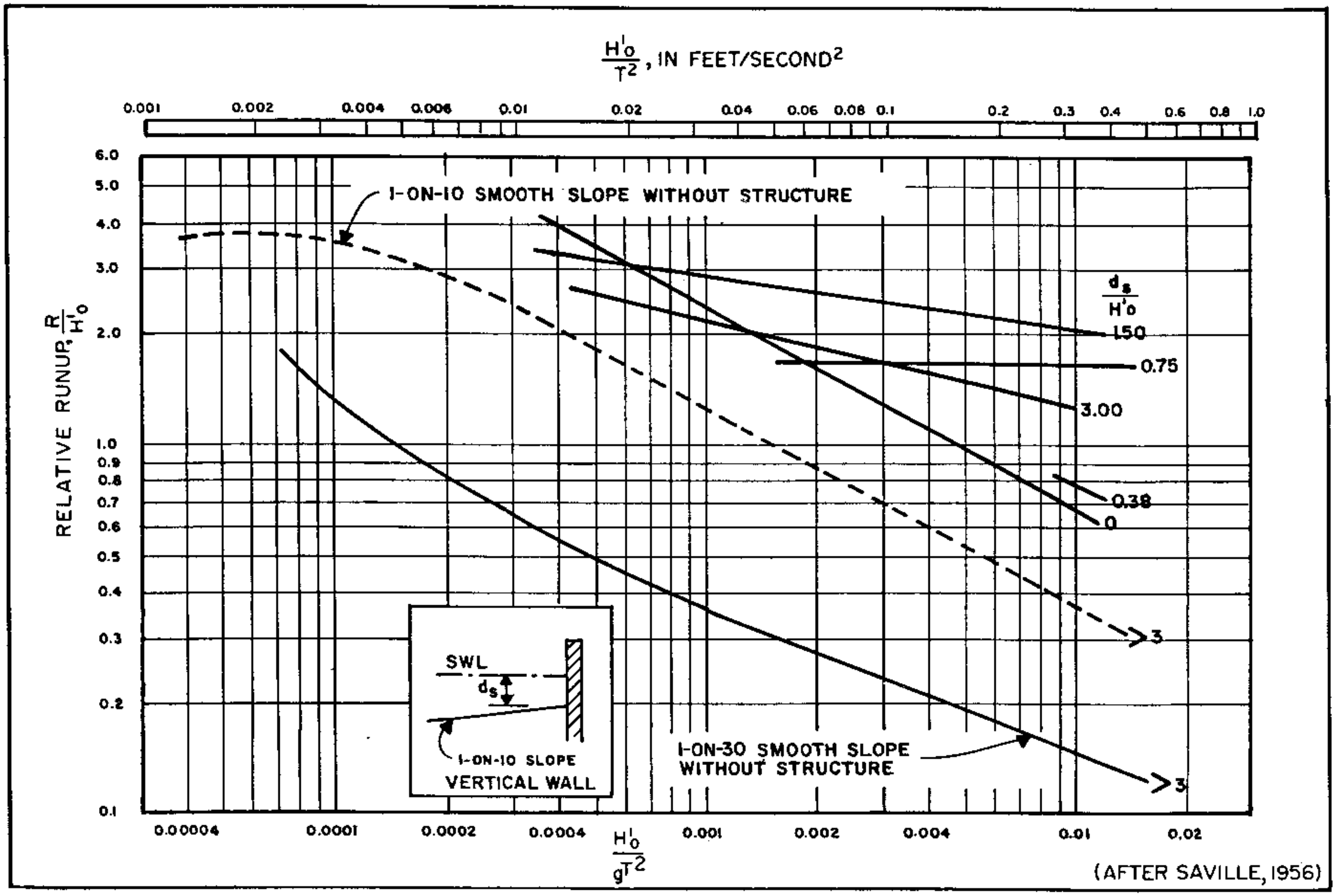


FIGURE 86
Relative Runup, R/H'_o, on an Impermeable Vertical Wall

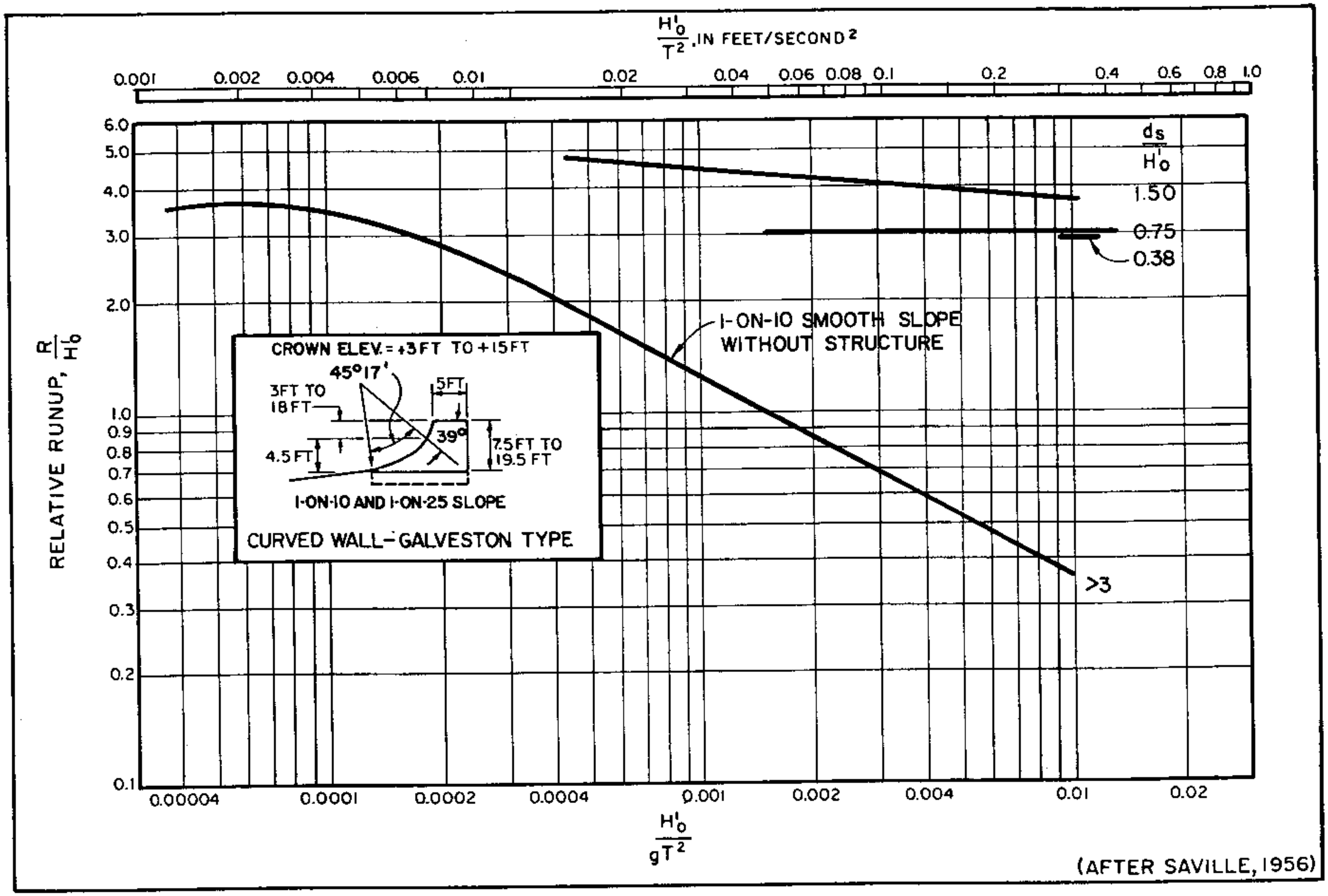


FIGURE 87
Relative Runup, R/H'_o, on a Curved Wall--Galveston Type

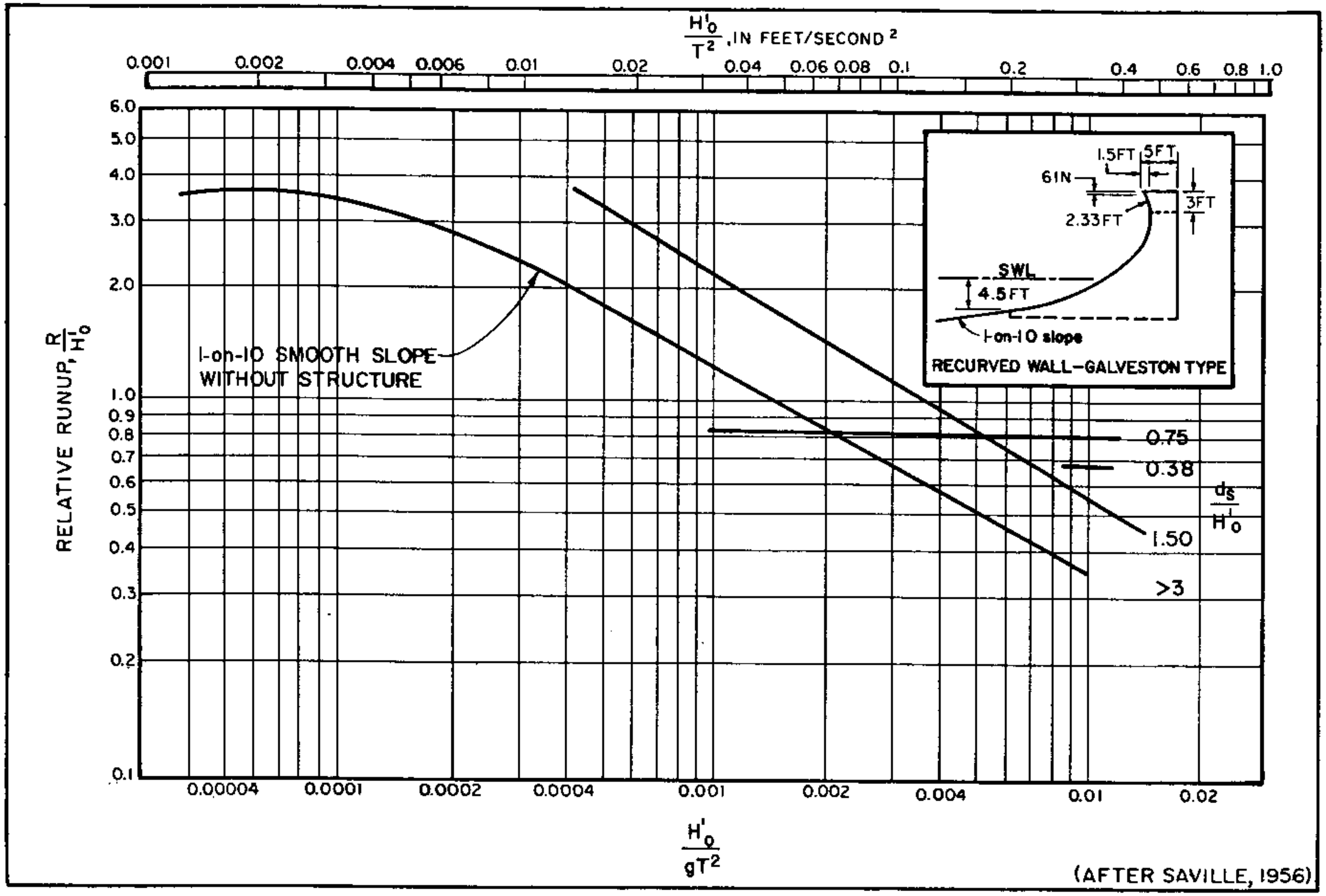


FIGURE 88

Relative Runup, R/H'_o, on a Recurved Wall--Galveston Type

EXAMPLE PROBLEM 22 (Continued)

$$\frac{H'_o}{g T^2} = \frac{3}{(32.2)(3)^2} = 0.0104$$

(3) From Figure 88 for $d_s/H'_o = 1.5$ and $H'_o/g T^2 = 0.0104$:

$$\frac{R}{H'_o} = 0.54$$

(4) Using Equation (3-11), find R:

$$R = (H'_o)(R/H'_o)$$

$$R = (3)(0.54) = 1.6 \text{ feet}$$

$$R = 1.6 \text{ feet}$$

(7) Case 8: Beach Slopes. Wave runup on sloping sandy beaches can be calculated by using the smooth-slope curves in Figures 72 through 81 to find R/H'_o, and by applying the appropriate runup scale-effect correction factor, k, from Figure 89. The rough-slope runup correction factor, r, is 1.00.

$$R = (H'_o)(R/H'_o)(r)(k)$$
$$R = (H'_o)(R/H'_o)(k) \qquad (3\text{-}12)$$

WHERE: k is found from Figure 89

EXAMPLE PROBLEM 23

Given:
a. Equivalent unrefracted deepwater wave height, $H'_o = 5$ feet
b. Water depth at toe of slope, $d_s = 15$ feet
c. Wave period, T = 3 seconds
d. Beach slope, cot [theta] = 20

Find: Runup on beach face.

Solution: (1) Find d_s/H'_o:

$$\frac{d_s}{H'_o} = \frac{15}{5} = 3; \text{ therefore, use Figure 72}$$

(2) Find $H_o/g T^2$:

$$\frac{H'_o}{g T^2} = \frac{5}{(32.2)(3)^2} = 0.0173$$

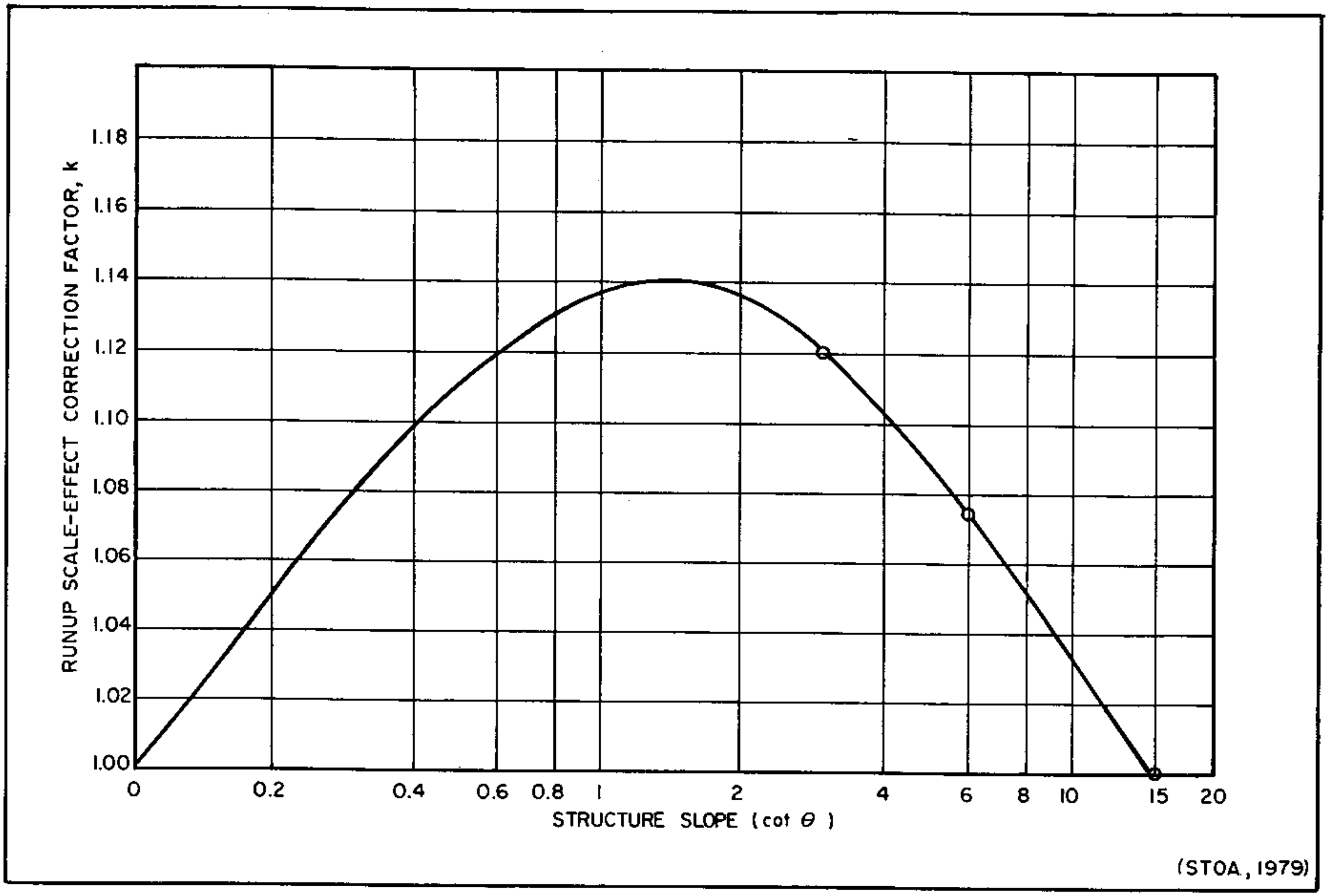


FIGURE 89
Runup Scale-Effect Correction Factor

EXAMPLE PROBLEM 23 (Continued)

(3) From Figure 72 for cot [theta] = 20 and $H'_o/g T^2$ = 0.0173:

$$\frac{R}{H'_o} = 0.15$$

(4) From Figure 89 for cot [theta] = 20:

k = 1.00

(5) Using Equation (3-12), find R:

$R = (H'_o) (R/H'_o) (k)$

R = (5) (0.15) (1.00) = 0.750 feet

R = 0.75 feet

(8) Special Precautions. Runup calculations are usually made for the significant wave height. It should be borne in mind that larger waves can impinge on the structure if the height is not limited by water depth. The runup that 1 percent of waves exceed can be significantly more than the runup due to the significant wave. Care should be exercised when designing the structure to determine the consequences of minor and occasional overtopping. The significant wave is generally adequate for most design situations. All beach slopes and structures do not necessarily fit the design curves given above. Volume II of the Shore Protection Manual (1977) should be consulted to determine the runup over composite slopes.

5. WAVE TRANSMISSION.

a. Design Parameters. Breakwaters are designed to attenuate waves propagating into the lee of the structure. Waves incident to a breakwater can be dissipated on, transmitted through, or transmitted over the structure. The amount of wave transmission depends on the incident wave height, H_i, the wave period, T, the water depth, d_s, the breakwater type (such as vertical-wall, vertical-thin wall, composite breakwater, wave-baffle, and rubble-mound), breakwater permeability, and breakwater geometry (crest height, h_s, crest width, b, and slope, cot [theta]).

The ratio of the transmitted, H_t, to incident, H_i, wave height is the transmission coefficient, K_t.

$$K_t = H_t/H_i \qquad (3\text{-}13)$$

WHERE: K_t = transmission coefficient

H_t = transmitted wave height

H_i = incident wave height

(1) Vertical-Wall, Vertical-Thin Wall, or Composite Breakwaters. These breakwaters are impermeable and transmission occurs by overtopping. The transmission is primarily a function of the incident wave height, H_i, the water depth at the structure, d_s, the crest width, b, the slope-protection depth, d_1, and the structure height, h_s. Figures 90 and 91 are used to determine transmission coefficients for impermeable structures. These figures are applicable over the range $0.015 \leq d_s/gT^2 \leq 0.0793$.

EXAMPLE PROBLEM 24

Given:
a. Deepwater significant wave height, H_o = 7 feet
b. Refraction coefficient, K_R = 0.9
c. Wave period, T = 4 seconds
d. Water depth, d_s = 25 feet

Find: Based on significant wave, determine the height of a thin-wall breakwater necessary to reduce the waves in its lee to 1.5 feet.

Solution: (1) Find H'_o:

$$H'_o = H_o K_R = (7)(0.9) = 6.3 \text{ feet}$$

(2) Find d_s/L_o :

$$L_o = (g/2\pi) T^2 = (32.2/2\pi)(4)^2 = 82.0 \text{ feet}$$

$$\frac{d_s}{L_o} = \frac{25}{82.0} = 0.305$$

(3) From Figure 2 for $d_s/L_o = 0.305$:

$$H/H'_o = 0.95$$

$$H_o = (0.95)(6.3) = 6.0 \text{ feet}$$

$$H = H_i = 6.0 \text{ feet}$$

(4) The desired transmitted wave height, H_t = 1.5 feet.

$$K_t = \frac{H_t}{H_i}$$

$$K_t = \frac{1.5}{6.0} = 0.25$$

(5) From Figure 90, using curve 1 and K_t = 0.25:

$$(h_s - d_s)/H_i = 0.5$$

$$(h_s - d_s) = 0.5\, H_i$$

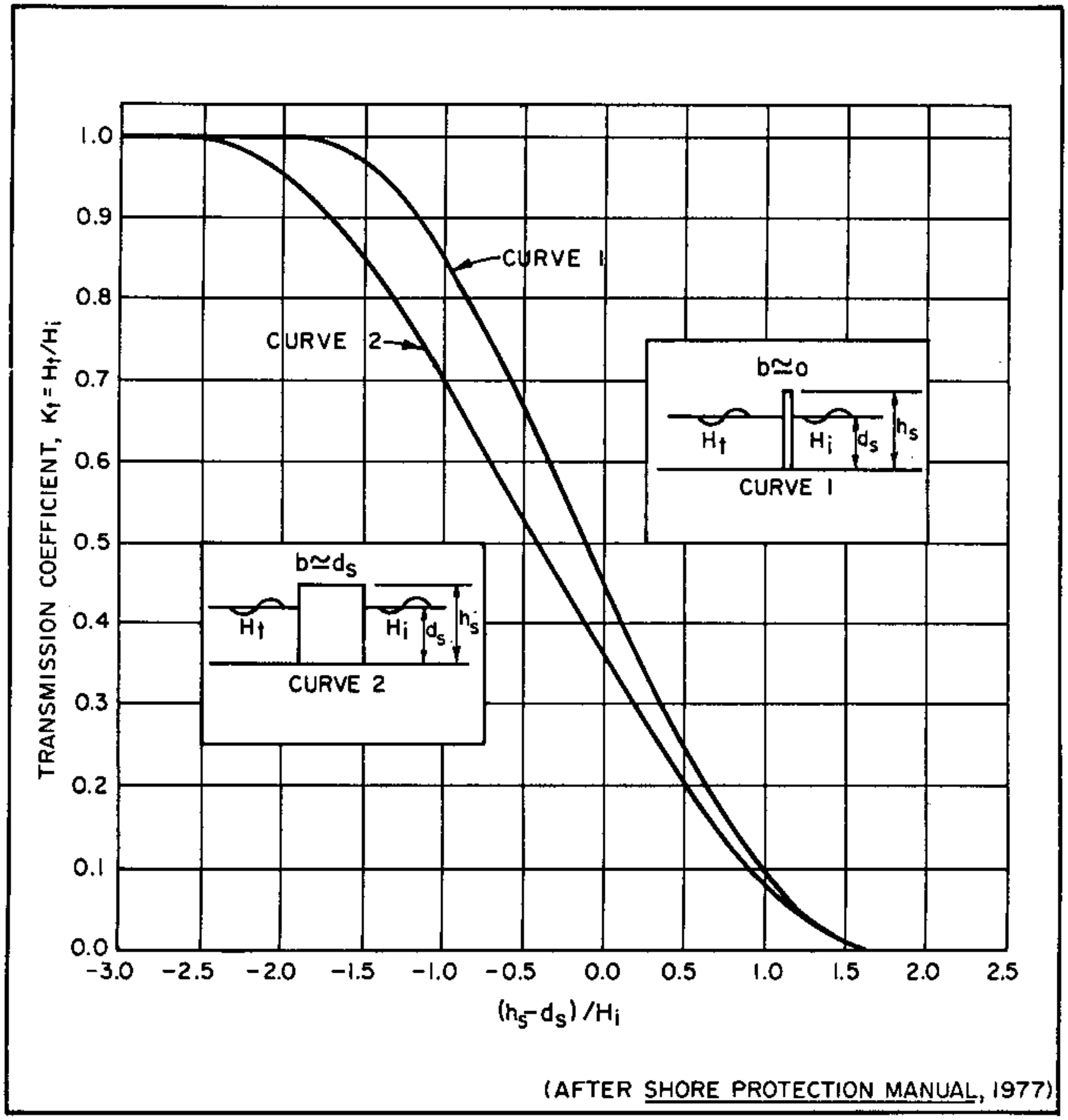


FIGURE 90
Transmission Coefficient, K_t, for Impermeable Structures
($0.015 \leq d_s/gT^2 \leq 0.0793$)

Structures (0.015 < / = dÚs¿/g TÀ2Ù < / = 0.0793)]

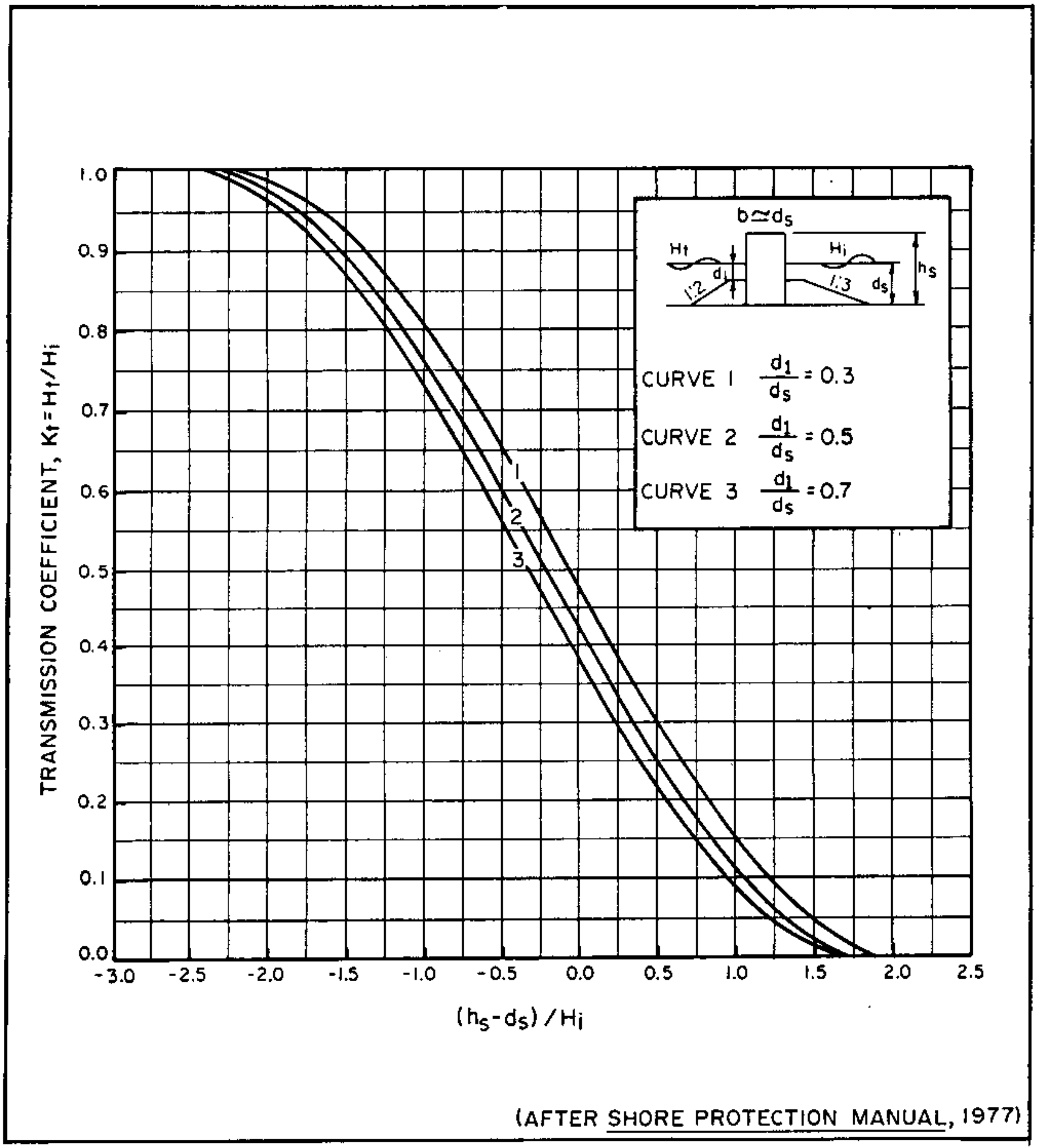


FIGURE 91
Transmission Coefficient, K_t, for Impermeable Structures
$(0.015 \leq d_s/gT^2 \leq 0.0793)$

Structures (0.015 < / = dÚs¿/g TÀ2Ù < / = 0.0793)]

EXAMPLE PROBLEM 24 (Continued)

$$h_s = 0.5\ H_i + d_s$$

$$h_s = (0.5)(6.0) + 25 = 28 \text{ feet}$$

$$h_s = 28 \text{ feet above the bottom}$$

(2) Wave-Baffle Breakwaters. A wave-baffle breakwater consists of a pile-supported impermeable barrier extending below the surface, but not extending down the entire depth of the water column. Prediction of the wave-transmission coefficient for this structure can be made using Figure 92; this figure predicts the transmission coefficient, K_t , as a function of the depth to wavelength ratio, d_s/L, and of the depth of baffle below the surface to depth ratio, h/d_s. Figure 92 accounts for the transmission of waves under the structure. If the wave baffle is of low height, then wave overtopping may occur in addition to transmission under the structure. In this case, complex interaction between the waves and the structure precludes easy determination of wave transmission, and a physical model study would be necessary.

(3) Rubble-Mound Breakwaters. Rubble-mound breakwaters are rough and permeable, and wave energy is dissipated on the front slope, transmitted over the structure, and transmitted through the voids of the structure. The transmitted wave height is primarily a function of the incident wave steepness, $H_i/g\ T^2$, the ratio of water depth to structure height, d_s/h_s, the ratio of incident wave height to water depth, h_s/d_s, the structure permeability, and the materials making up the breakwater under consideration. Other parameters of secondary importance include type of armor unit and placement, crest width, structure slope, and relative depth, $d_s/g\ T^2$. The problem is complex and it is generally necessary to resort to a physical or mathematical model to determine wave-transmission characteristics if the design situation is critically dependent upon transmitted waves. The procedures described subsequently can be used as a first approximation. These procedures are based on methods given in Seelig (1980) which use a computer model for the specific breakwater cross sections shown in Figure 93; these cross sections represent typical designs for the given range of water depths and wave conditions. Where breakwater construction differs significantly from that shown for each water depth, wave transmission should be determined by using procedures given in Seelig (1980) or by using a physical model. An example using the method described below is presented in Section 4, Example Problem 27.

(a) Structures in shallow water. Shallow-water breakwaters are usually relatively permeable. The core is generally low and most of the breakwater is made up of high-void armor units. Figure 94 can be used to determine the transmission coefficient, K_t, for a given $H_i/g\ T^2$, d_s/h_s, and H_i/d_s. The curves are based on analysis of the shallow-water structure shown in Figure 93 and are applicable for design water depths ranging from 5 to 10 feet.

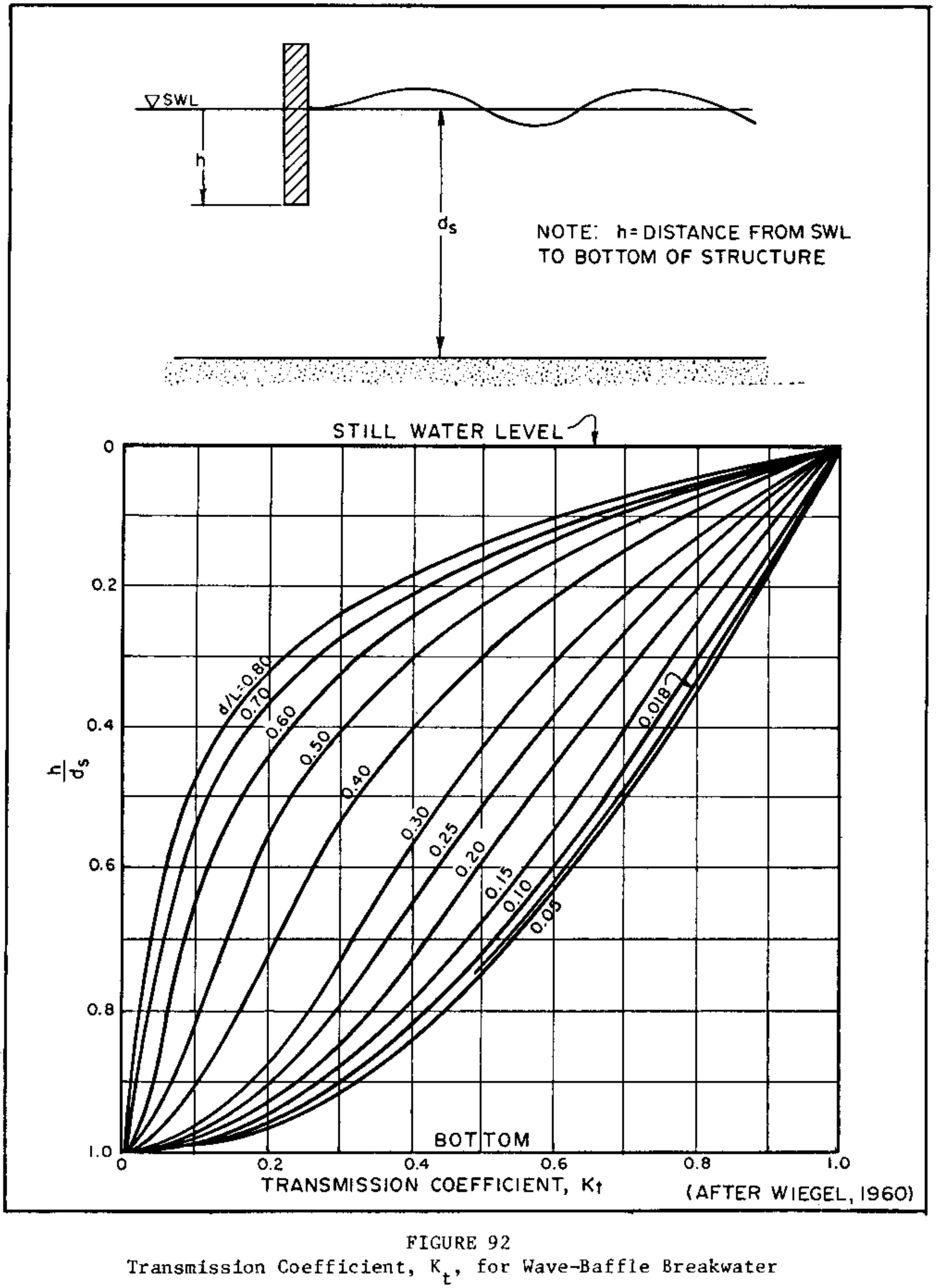


FIGURE 92
Transmission Coefficient, K_t, for Wave-Baffle Breakwater

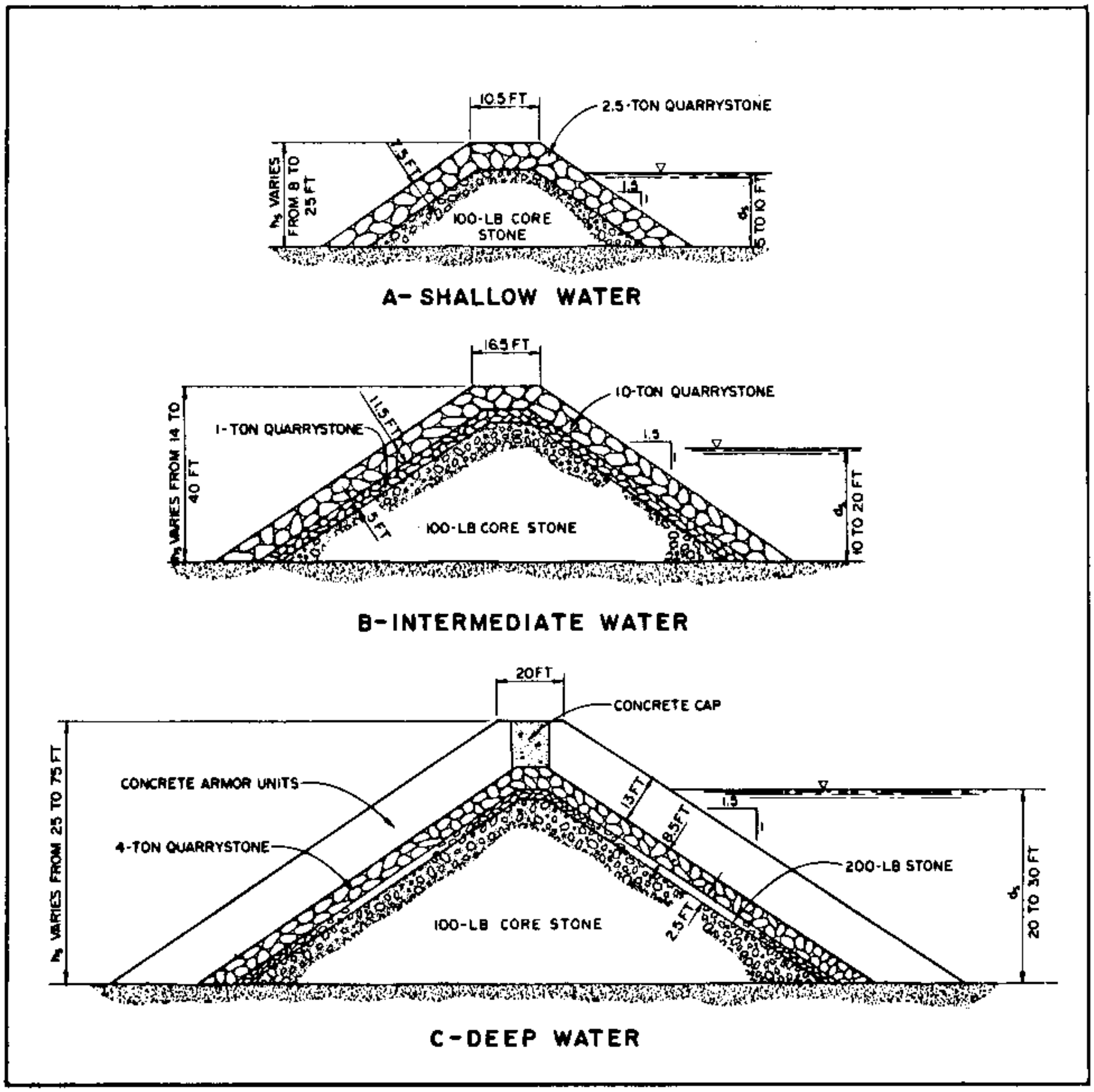


FIGURE 93
Typical Breakwater Cross Sections Used to Determine Wave Transmission

Transmission]

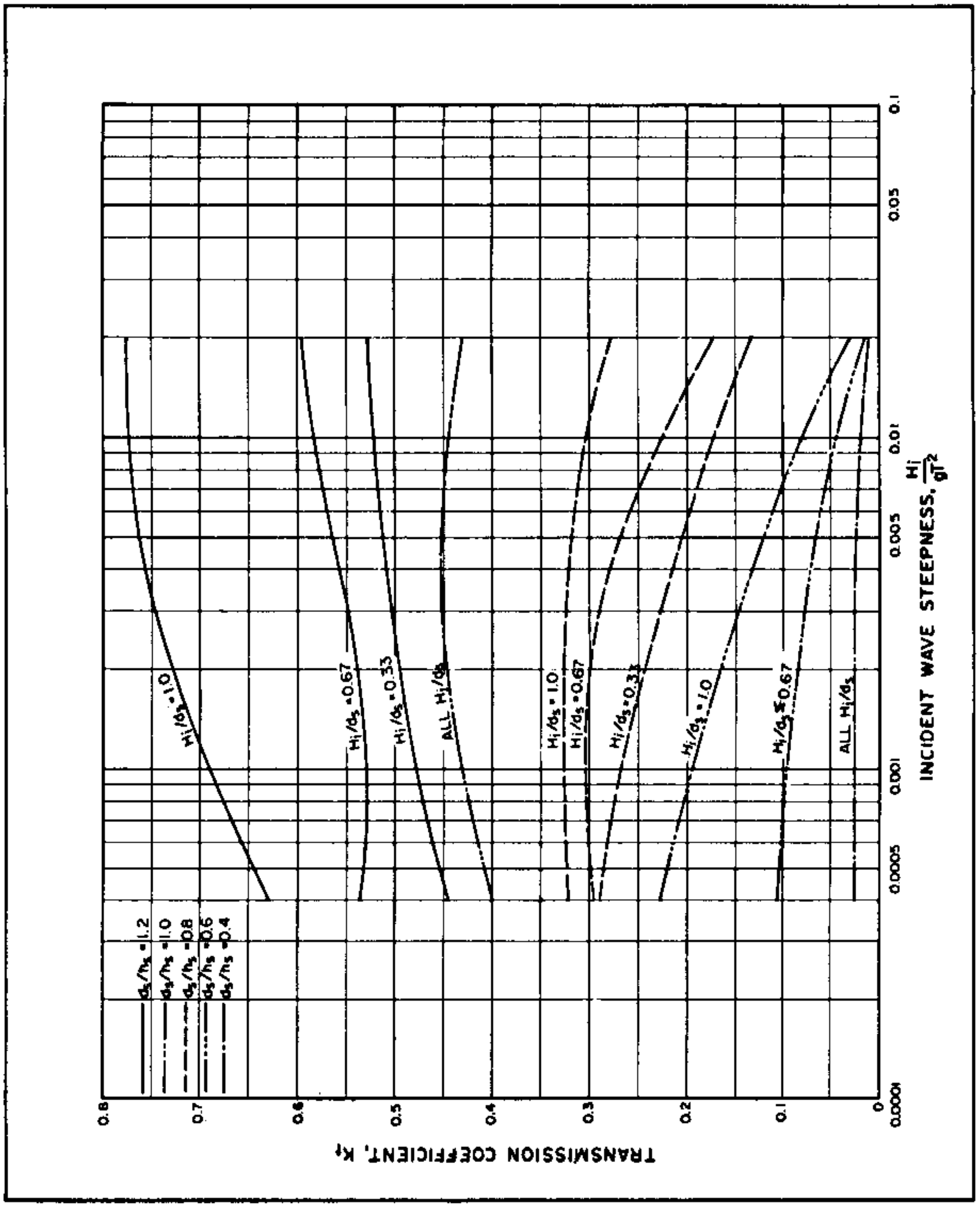


FIGURE 94
Transmission Coefficient, K_t, for a Typical Breakwater in Depths, d_s, Ranging From 5 to 10 Feet

in Depths, dÚs¿, Ranging From 5 to 10 Feet]

(b) Structures in intermediate water depths. Rubble-mound structures built in intermediate design water depths (10 to 20 feet) are typically multilayered breakwaters with a relatively high core of well-graded quarry run. Transmission coefficients may be determined using Figure 95. This figure is based on analysis of the intermediate-depth-water structure shown in the Figure 93.

(c) Structures in deep water. Structures constructed in deep water (20 to 30 feet) usually have large, relatively impermeable, cores of well-graded material. The armor-layer thickness is small compared to structure height and typically is made up of concrete armor units. Figure 96 is used to determine transmission coefficients based on the deep-water structure shown in Figure 93. Breakwaters constructed in water deeper than 30 feet should be analyzed by using a physical model.

6. WAVE REFLECTION.

a. Discussion. In certain design applications, wave reflection from structures or beaches should be considered. This is true of situations where reflection will have an adverse effect on navigation or littoral processes. The type of reflection pattern present depends on the angle of incidence, [alpha]. The angle of reflection is denoted r.

Two basic patterns of wave reflection take place when waves approach a structure at an angle: "normal" reflection or "Mach" reflection. The critical angle of incidence separating the two patterns is 45 deg. For [alpha] > 45 deg., the reflection pattern is "regular;" for [alpha] < 45 deg., the reflection pattern is the "Mach" reflection type. The two types of wave reflection are summarized in Table 9 and diagrammed in Figure 97.

TABLE 9
Types of Wave Reflection

Reflection Type	Angle of Incidence	Reflection Pattern
Regular	90 deg. > [alpha] > 45 deg.	1. [alpha] = r
Mach	[alpha] < 20 deg.	1. No reflected wave 2. Mach-stem wave perpendicular to wall
	45 deg. > [alpha] > 20 deg.	1. [alpha] < r 2. Mach-stem wave perpendicular to wall

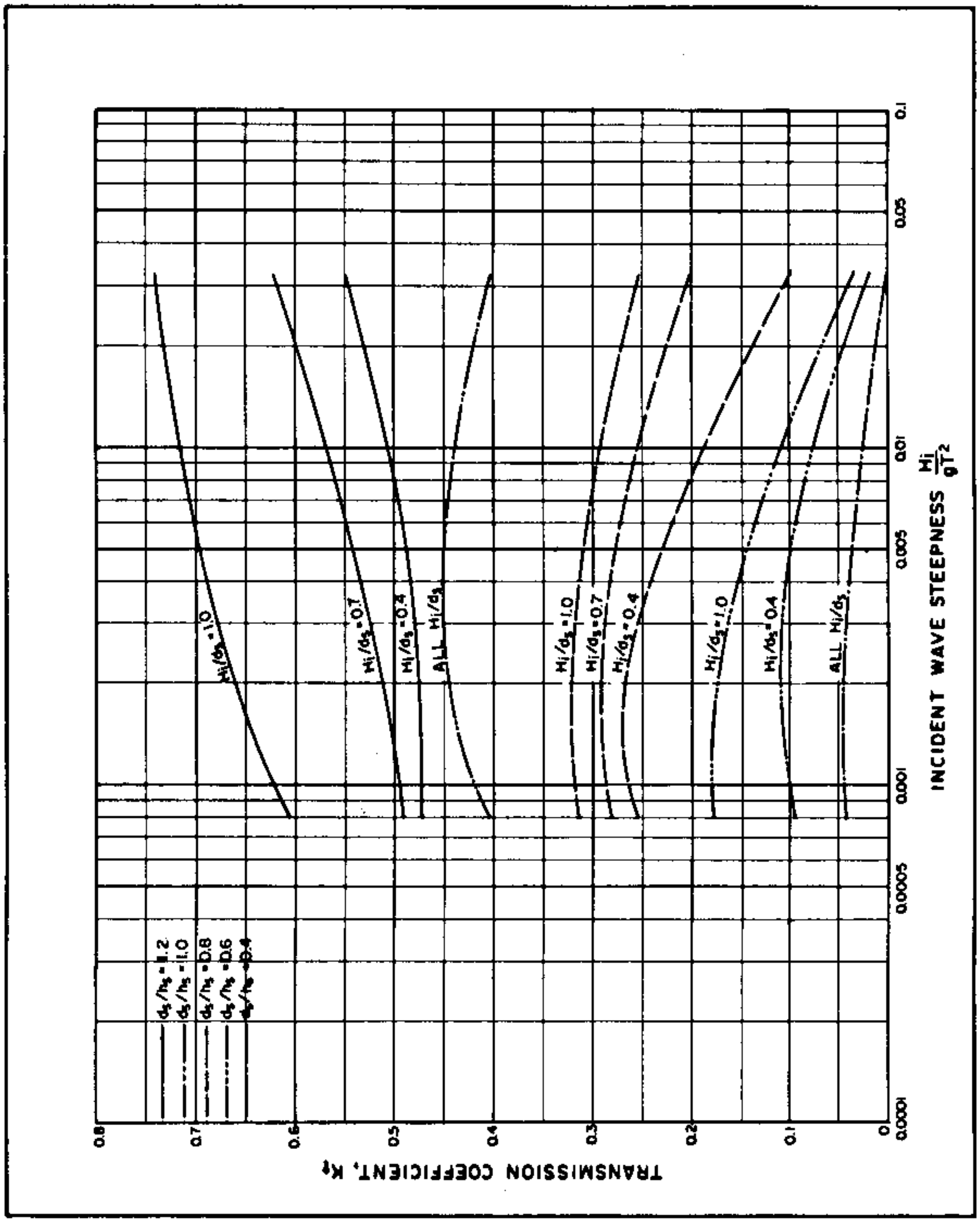


FIGURE 95
Transmission Coefficient, K_t, for a Typical Breakwater in Depths, d_s, Ranging From 10 to 20 Feet

in Depths, dÚs¿, Ranging From 10 to 20 Feet]

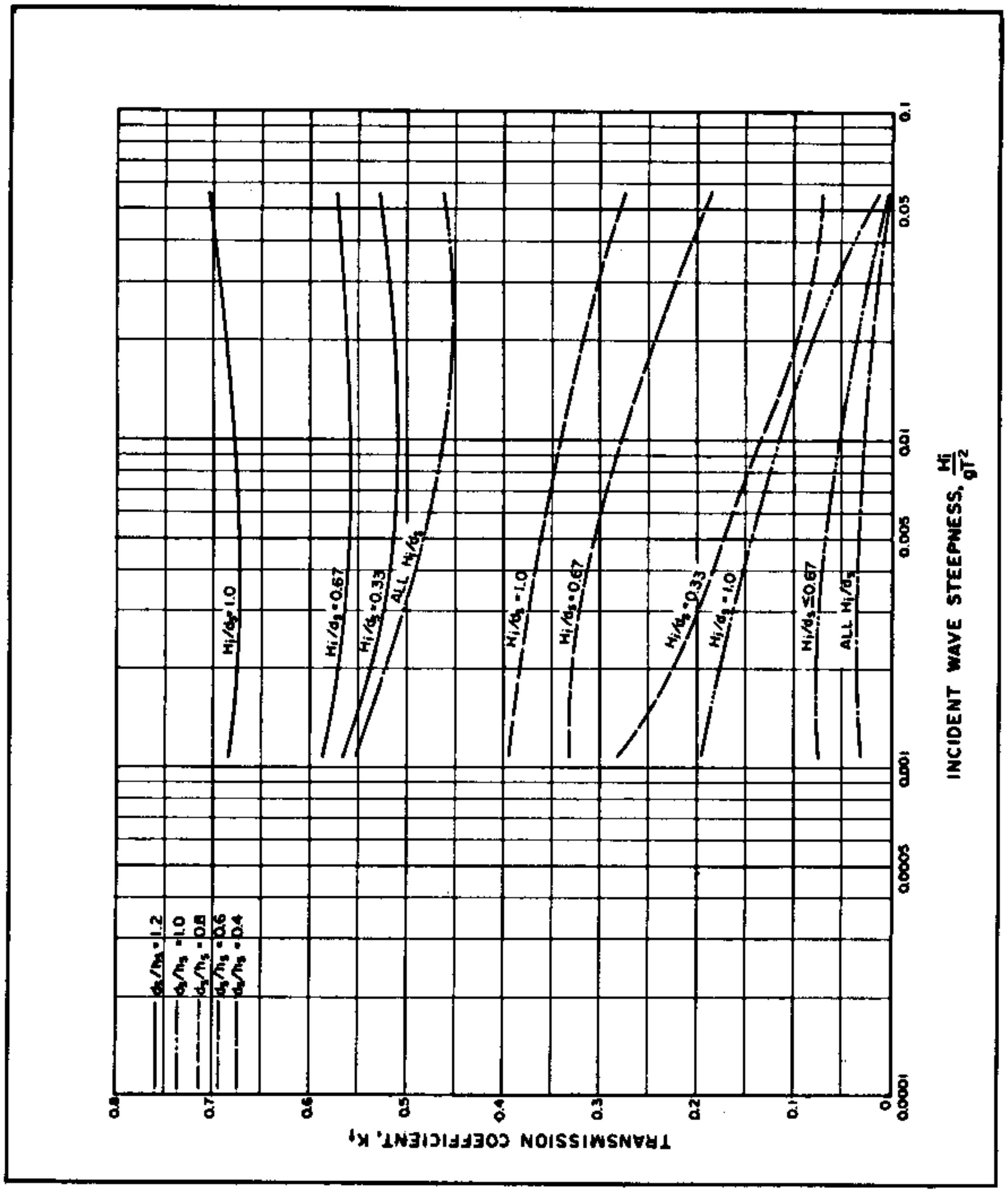


FIGURE 96
Transmission Coefficient, K_t, for a Typical Breakwater in Depths, d_s, Ranging From 20 to 30 Feet

in Depths, dÚs¿, Ranging From 20 to 30 Feet]

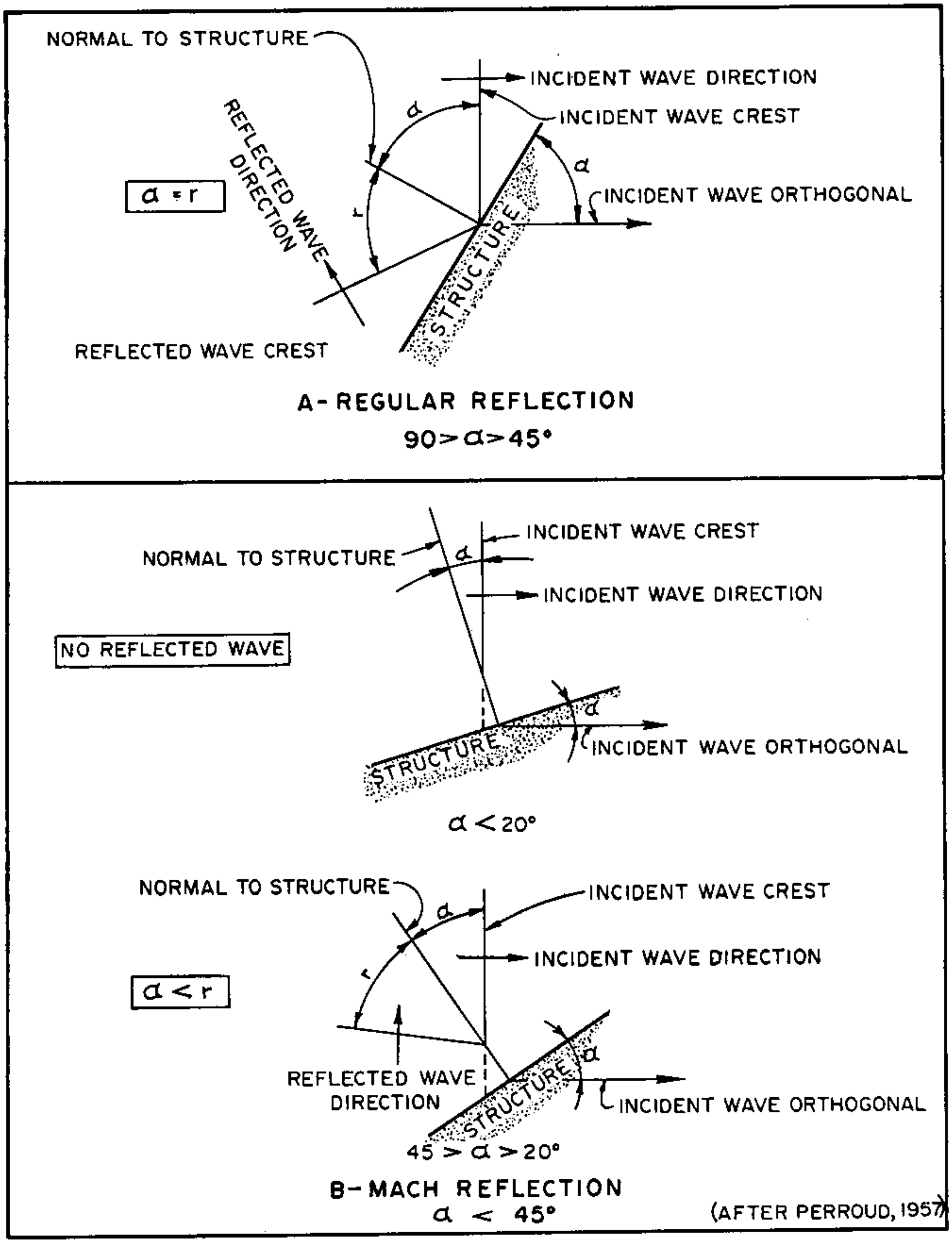


FIGURE 97
Definition of Terms Used in Reflection Calculations

For "regular" reflection (when [alpha] > 45 deg.), the angle of incidence equals the angle of reflection, and the reflected wave height, HÚr¿, is only slightly less than the incident wave height, HÚi¿. For "Mach" reflection, there are two subcases of reflection pattern. For [alpha] < 20 deg., the incident wave crest bends so that it becomes perpendicular to the structure, and no reflected wave appears. For [alpha] > 20 deg. (but < 45 deg.), three waves are present: the incident wave, the reflected wave, and the "Mach-stem" wave (the portion of the wave crest perpendicular to the structure). For the reflected wave, the height, HÚr, is smaller than the incident wave height, HÚi¿, and the angle of reflection is greater than the angle of incidence. The wave height of the Mach-stem wave, HÚm¿, is greater than the incident wave height, HÚi¿.

b. Calculation of Reflection. Wave reflection from revetments, rubble-mound breakwaters, and beaches is determined through empirical equations developed by Seelig and Ahrens (1980) for waves approaching a structure at normal incidence. Figure 98 is a sketch defining terms used in reflection calculations. The general equation for determining reflected wave height is:

$$K_r = \frac{\alpha \xi^2}{\beta + \xi^2} \tag{3-14}$$

WHERE: K_r = reflection coefficient, defined as the ratio of the reflected, H_r, to the incident, H_i, wave height = H_r/H_i

α, β = empirically derived constants which depend on structure type, water depth at structure toe, roughness of slope, number of armor layers, and breaking-wave height

$\xi = \dfrac{\tan\theta}{\sqrt{\dfrac{H_i}{L_o}}}$ = surf parameter

θ = structure-slope angle

L_o = deepwater wavelength

(1) Revetments.

(a) Smooth-face revetments, $d_s/H_i > 5$. In this situation, the interaction of the incident wave with, or modification of the incident wave by, the presence of the structure dominates the magnitude of the reflection coefficient. For values of $\tan\theta \leq 0.1667$, use $\alpha = 1.0$ and $\beta = 5.5$; thus, Equation (3-14) becomes:

$$K_r = \frac{1.0\ \xi^2}{5.5 + \xi^2} \tag{3-15}$$

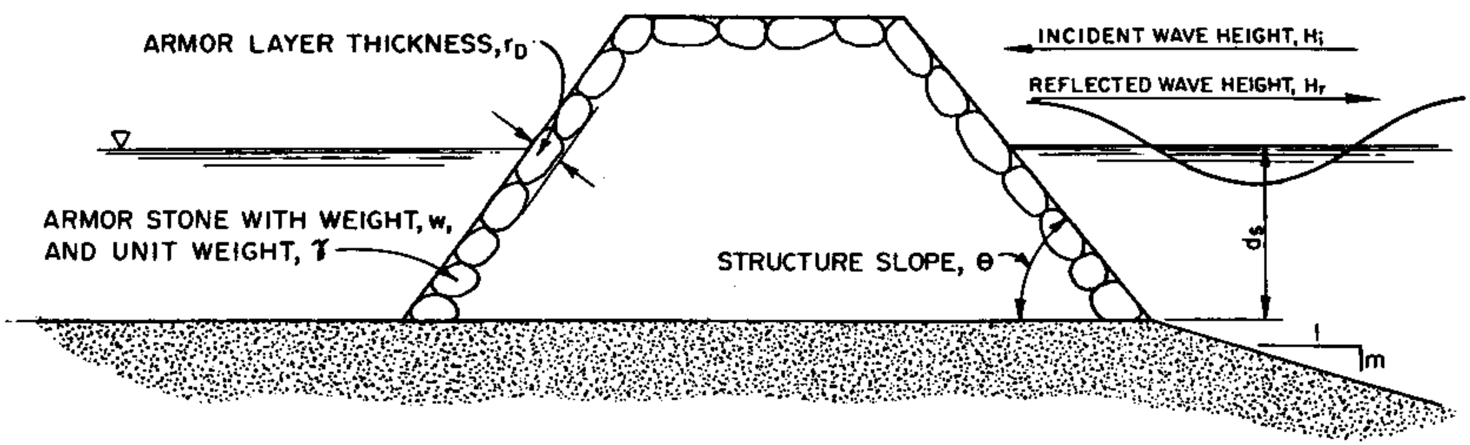


FIGURE 98
Terms Used in Reflection Calculations

(b) Smooth-face revetments, d_s/H_i < / = 5. In this situation, the reflection coefficient is influenced by the wave breaking at the toe of or seaward of the structure. The value of [alpha] for use in Equation (3-14) is defined as:

$$[\alpha] = \exp \left[(-0.5) \left(\frac{H_i}{H_b}\right)^{1/3} \right] \tag{3-16}$$

WHERE: exp = e = base of natural logarithm = 2.718...

H_i = incident wave height

$$H_b = 0.17\, L_o \left\{1 - \exp \left[-4.712 \left(\frac{d_s}{L_o}\right)(1 + 15\, m^{1.333}) \right] \right\} \tag{3-17}$$

= breaking-wave height

d_s = water depth at structure toe

L_o = deepwater wavelength

m = slope in front of structure

The value for [BETA] is 5.5.

(c) Rubble-mound revetments with one layer of armor stone. In this situation, the presence of a rough surface will reduce the amount of wave reflection. The value of [alpha] for use in Equation (3-14) is defined as:

$$\alpha = \exp \left[-1.7\sqrt{\frac{r_D}{L}} \cot \theta - (0.5)\left(\frac{H_i}{H_b}\right)^{1.3}\right] \tag{3-18}$$

WHERE: exp = e = base of natural logarithm = 2.718...

r_D = $(w/\gamma)^{1/3}$ = representative armor-stone diameter

w = armor-stone weight

γ = unit weight of armor stone

L = wavelength at structure toe

θ = structure-slope angle

H_i = incident wave height

H_b = breaking-wave height (Equation 3-17)

The value for β is 5.5.

$$\alpha = \alpha' \exp\left[-1.7 \sqrt{\frac{r_D}{L}} \cot\theta - (0.5)\left(\frac{H_i}{H_b}\right)^{1.3}\right] \qquad (3\text{-}19)$$

WHERE: α' is found in Table 10

exp = e = base of natural logarithm = 2.718...

r_D = representative armor-stone diameter

L = wavelength at structure toe

θ = structure-slope angle

The value for β is 5.5.

TABLE 10
Reflection-Coefficient Reduction Factor, [alpha]',
Due to Multiple Armor Layers

Number of Layers	2	3	4
r_D / H_i			
< 0.75	0.93	0.88	0.78
0.75 to 2.0	0.71	0.70	0.69
> 2.0	0.58	0.52	0.49

(SEELIG AND AHRENS, 1980)

EXAMPLE PROBLEM 25

Given:
a. Incident wave height, H_i = 10 feet
b. Water depth at structure toe, d_s = 15 feet
c. Wave period, T = 8 seconds
d. Structure slope, $\tan\theta = 0.667$
e. Armor-stone weight, w = 3 tons
f. Unit weight of armor stone, γ = 160 pounds per cubic foot
g. Slope in front of structure, m = 1/20

Find: Height of reflected wave for structure with one layer of armor.

Solution: (1) Find L_o:

$$L_o = (g/2\pi)\, T^2 = (32.2/2\pi)(8)^2 = 327.7 \text{ ft}$$

(2) Using Equation (3-17), find H_b:

$$H_b = 0.17\, L_o \left\{1 - \exp\left[-4.712\left(\frac{d_s}{L_o}\right)(1 + 15\, m^{1.333})\right]\right\}$$

$$H_b = (0.17)(327.7)\left\{1 - \exp\left\{-4.712\left(\frac{15}{327.7}\right)\left[1 + (15)\left(\frac{1}{20}\right)^{1.333}\right]\right\}\right\}$$

$H_b = 13.41$ feet

(3) Convert armor-stone weight, in tons, to pounds:

$$(3 \text{ tons})\left(\frac{2{,}000 \text{ pounds}}{1 \text{ ton}}\right) = 6{,}000 \text{ pounds}$$

(4) Find r_D:

$$r_D = \left(\frac{w}{\gamma}\right)^{1/3} = \left(\frac{6{,}000}{160}\right)^{1/3} = 3.35 \text{ feet}$$

(5) Find L:

$$\frac{d_s}{L_o} = \frac{15}{328} = 0.0457$$

From Figure 2 for $d_s/L_o = 0.0457$:

$$\frac{d_s}{L} = 0.09$$

$$L = \frac{d_s}{0.09} = \frac{15}{0.09} = 166.7 \text{ feet}$$

(6) Using Equation (3-18), find α:

$$\alpha = \exp\left[-1.7\sqrt{\frac{r_D}{L}}\cot\theta - (0.5)\left(\frac{H_i}{H_b}\right)^{1.3}\right]$$

$$\alpha = \exp\left[-1.7\sqrt{\frac{3.35}{166.7}}(1.5) - (0.5)\left(\frac{10}{13.4}\right)^{1.3}\right]$$

$$\alpha = 0.49$$

(7) Find ξ :

$$\xi = \frac{\tan\theta}{\sqrt{\frac{H_i}{L_o}}} = \frac{0.667}{\sqrt{\frac{10}{328}}} = 3.82$$

(8) $\beta = 5.5$

(9) Using Equation (3-14), find K_r:

$$K_r = \frac{\alpha\,\xi^2}{\beta + \xi^2} = \frac{(0.49)(3.82)^2}{5.5 + (3.82)^2} = 0.36$$

(10) Find H_r:

EXAMPLE PROBLEM 25 (Continued)

$$K_r = \frac{H_r}{H_i}$$

THEREFORE: $H_r = K_r H_i = (0.36)(10) = 3.6$ feet

(2) Rubble-Mound Breakwaters. A conservative estimate of K_r for rubble-mound breakwaters armored with quarrystone or dolosse is determined as follows:

$$K_r = \frac{0.6\,\xi^2}{6.6 + \xi^2} \tag{3-20}$$

EXAMPLE PROBLEM 26

Given: a. Incident wave height, $H_i = 20$ feet
b. Water depth at structure toe, $d_s = 35$ feet
c. Wave period, $T = 15$ seconds
d. Structure slope, $\tan\theta = 0.5$

Find: Reflected wave height from a rubble-mound breakwater.

Solution: (1) Find L_o:

$$L_o = (g/2\pi)\,T^2 = (32.2/2\pi)(15)^2 = 1{,}153.1 \text{ feet}$$

(2) Find ξ:

$$\xi = \frac{\tan\theta}{\sqrt{\frac{H_i}{L_o}}} = \frac{0.5}{\sqrt{\frac{20}{1{,}153.1}}} = 3.797$$

(3) Using Equation (3-20), find K_r:

$$K_r = \frac{0.6\,\xi^2}{6.6 + \xi^2}$$

$$K_r = \frac{(0.6)(3.797)^2}{6.6 + (3.797)^2} = 0.412$$

(4) Find H_r:

$$K_r = \frac{H_r}{H_i}$$

THEREFORE: $H_r = K_r H_i = (0.412)(20) = 8.24$ feet

EXAMPLE PROBLEM 26

Given:
a. Incident wave height, H_i = 20 feet
b. Water depth at structure toe, d_s = 35 feet
c. Wave period, T = 15 seconds
d. Structure slope, $\tan \theta = 0.5$

Find: Reflected wave height from a rubble-mound breakwater.

Solution: (1) Find L_o:

$$L_o = (g/2\pi)\ T^2 = (32.2/2\pi)(15)^2 = 1{,}153.1 \text{ feet}$$

(2) Find ξ:

$$\xi = \frac{\tan \theta}{\sqrt{\frac{H_i}{L_o}}} = \frac{0.5}{\sqrt{\frac{20}{1{,}153.1}}} = 3.797$$

(3) Using Equation (3-20), find K_r:

$$K_r = \frac{0.6\ \xi^2}{6.6 + \xi^2}$$

$$K_r = \frac{(0.6)(3.797)^2}{6.6 + (3.797)^2} = 0.412$$

(4) Find H_r:

$$K_r = \frac{H_r}{H_i}$$

THEREFORE: $H_r = K_r\ H_i = (0.412)(20) = 8.24$ feet

(3) Beaches. A conservative estimate of KÚr¿ for beaches is determined as follows:

```
           1.0 [xi]À2Ù
KÚr¿ = ÄÄÄÄÄÄÄÄÄÄÄ                    (3-21)
           5.5 [xi]À2Ù
```

In this situation, the value of tan [theta] for the beach is taken at the still water line.

7. METRIC EQUIVALENCE CHART. The following metric equivalents were developed in accordance with ASTM E-621. These units are listed in the sequence in which they appear in the text in Sect ion 3. Conversions are approximate.

5 feet = 1.5 meters
10 feet = 3.0 meters
20 feet = 6.1 meters
30 feet = 9.1 meters

SECTION 4. DESIGN OF RUBBLE-MOUND STRUCTURES

1. GENERAL. Rubble-mound structures generally have a core covered with one to several quarrystone underlayers which are protected with armor units of stone or specially shaped concrete units. Breakwaters have a core material of randomly dumped, well-graded quarry run, sand, or coral. This material is generally impermeable. Successive underlayers cover the core; the material in each successive layer is carefully increased in size to prevent loss of the smaller-sized core material. Armor units are placed on the outer surface to hold the core and the underlayers in place against wave attack. Rubblemound revetments, groins, and jetties are similarly built in that armor units hold the underlying material in place. Rubble-mound structures are well-suited to the coastal zone because they can absorb the forces of waves with relatively minor damage even when design conditions are exceeded to a moderate degree. Table 11 summarizes principles that should be considered in the design of rubble-mound structures.

2. DESIGN.

a. Armor Units. Generally, quarrystone is the most cost-effective armor unit; however, the use of concrete armor units, such as tetrapods, dolosse, and tribars (see Figures 99, 100, and 101, respectively), may be advantageous when nearby quarries cannot economically produce large enough stone. The weight of an individual armor unit can be found by the equation:

$$W = (w_r H^3)/[K_D (S_r - 1)^3 \cot \theta] \qquad (4\text{-}1)$$

WHERE: W = weight of individual armor unit

w_r = unit weight of armor material (saturated surface dry)

H_o = design wave height (use significant wave height, H_s)

K_D = stability coefficient (see Table 12 for appropriate value)

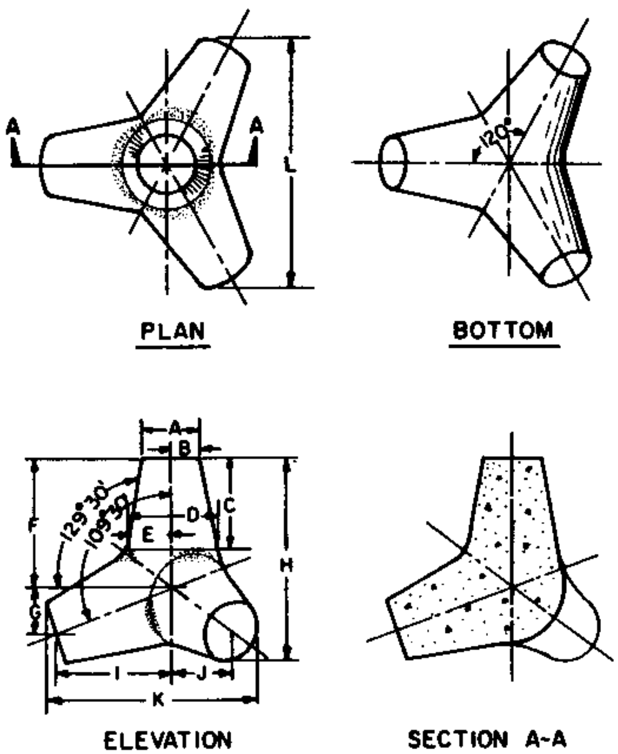
$S_r = w_r/w_w$ = specific gravity of armor unit, based on the unit weight of water at the structure

w_w = unit weight of water

θ = angle of structure slope measured relative to the horizontal

b. Design Considerations. The following should be carefully considered:

(1) Structure slopes should not be steeper than 1:1.5 (1:2 where dolosse are used), without conducting model studies. In addition to selection of proper armor units, other considerations should be investigated in rubble-mound design, such as possible settlement of the structure and stability of the soil supporting the structure against slope failure.



Unit Weight (LB/CU FT)											
	VOLUME OF INDIVIDUAL ARMOR UNITS (CU FT)										
	7.14	14.29	28.57	71.43	142.86	214.29	285.71	357.14	428.57	500.00	571.43
UNIT WEIGHT (LB/CU FT)	**WEIGHT OF INDIVIDUAL ARMOR UNITS (TONS)**										
140.0	0.50	1.00	2.00	5.00	10.00	15.00	20.00	25.00	30.00	35.00	40.00
149.5	0.53	1.07	2.14	5.34	10.68	16.02	21.36	26.70	32.04	37.38	42.71
156.0	0.56	1.11	2.23	5.57	11.14	16.71	22.29	27.86	33.43	39.00	44.57
162.0	0.58	1.16	2.31	5.79	11.57	17.36	23.14	28.93	34.71	40.50	46.29
	AVERAGE MEASURED THICKNESS OF TWO LAYERS, RANDOMLY PLACED (FT)										
	4.01	5.05	6.36	8.63	10.87	12.45	13.70	14.76	15.68	16.51	17.26
	NUMBER OF ARMOR UNITS PER 1,000 SQUARE FEET (TWO LAYERS, RANDOMLY PLACED)										
	280.18	176.49	111.18	60.42	37.96	29.02	24.02	20.70	18.41	16.54	15.18
SYMBOL	**DIMENSIONS OF ARMOR UNITS (FT)**										
A	0.89	1.12	1.41	1.91	2.41	2.76	3.04	3.27	3.48	3.66	3.83
B	0.44	0.56	0.70	0.95	1.20	1.36	1.52	1.63	1.74	1.83	1.91
C	1.40	1.77	2.23	3.02	3.81	4.36	4.80	5.17	5.50	5.79	6.05
D	1.38	1.74	2.20	2.96	3.76	4.30	4.73	5.10	5.42	5.70	5.96
E	0.69	0.87	1.10	1.49	1.88	2.15	2.37	2.55	2.71	2.85	2.96
F	1.89	2.36	3.00	4.08	5.14	5.88	6.47	6.97	7.41	7.80	8.16
G	0.63	0.79	1.00	1.36	1.71	1.96	2.16	2.32	2.47	2.60	2.72
H	2.94	3.71	4.67	6.34	7.99	9.14	10.07	10.84	11.52	12.13	12.68
I	1.78	2.25	2.83	3.84	4.84	5.54	6.10	6.57	6.98	7.35	7.69
J	0.89	1.12	1.41	1.92	2.42	2.77	3.05	3.28	3.49	3.67	3.84
K	3.21	4.04	5.09	6.91	8.71	9.97	10.97	11.82	12.56	13.23	13.83
L	3.54	4.45	5.61	7.62	9.60	10.96	12.09	13.02	13.84	14.57	15.23

NOTE: DATA BASED ON TETRAPODS USED IN MODEL TESTS CONDUCTED AT THE U.S. ARMY CORPS OF ENGINEERS WATERWAYS EXPERIMENT STATION

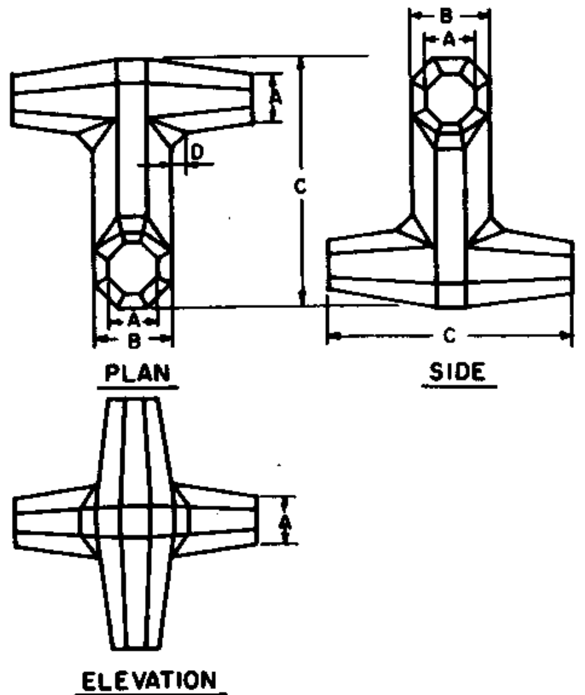
VOLUME OF INDIVIDUAL ARMOR UNIT = $0.280\ H^3$

WHERE: H = OVERALL DIMENSION OF UNIT

A = 0.302 H G = 0.215 H
B = 0.151 H I = 0.606 H
C = 0.477 H J = 0.303 H
D = 0.470 H K = 1.091 H
E = 0.235 H L = 1.201 H
F = 0.644 H

ARMOR-LAYER THICKNESS (2 UNITS) = 1.361 H

FIGURE 99
Tetrapod Specifications



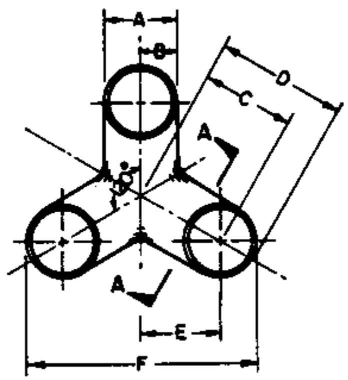
	VOLUME OF INDIVIDUAL ARMOR UNITS (CU FT)										
	7.14	14.29	28.57	71.43	142.86	214.29	285.71	357.14	428.57	500.00	571.43
UNIT WEIGHT (LB/CU FT)	WEIGHT OF INDIVIDUAL ARMOR UNITS (TONS)										
140.0	0.50	1.00	2.00	5.00	10.00	15.00	20.00	25.00	30.00	35.00	40.00
149.5	0.53	1.07	2.14	5.34	10.68	16.02	21.36	26.70	32.04	37.38	42.71
156.0	0.56	1.11	2.23	5.57	11.14	16.71	22.29	27.86	32.43	39.00	44.57
162.0	0.58	1.16	2.31	5.79	11.57	17.36	23.14	28.93	34.71	40.50	46.29
	AVERAGE MEASURED THICKNESS OF TWO LAYERS, RANDOMLY PLACED (FT)										
	3.6	4.6	5.7	7.8	9.8	11.2	12.4	13.3	14.2	14.9	15.6
	NUMBER OF ARMOR UNITS PER 1,000 SQUARE FEET (TWO LAYERS, RANDOMLY PLACED)										
	223.0	104.5	88.5	48.1	30.3	23.1	19.1	16.4	14.6	13.1	12.0
SYMBOL	DIMENSIONS OF ARMOR UNITS (FT)										
A	0.71	0.89	1.13	1.53	1.93	2.20	2.43	2.61	2.78	2.92	3.06
B	1.13	1.43	1.80	2.44	3.08	3.52	3.88	4.18	4.44	4.68	4.89
C	3.55	4.47	5.63	7.64	9.63	11.02	12.13	13.08	13.89	14.62	15.29
D	0.20	0.25	0.32	0.43	0.55	0.62	0.69	0.74	0.79	0.83	0.87

NOTE: SHAPE AND DIMENSIONS OF UNIT BASED ON THOSE USED IN MODEL TESTS. AT PRESENT TIME, INVENTOR RECOMMENDS B = 0.32 C TO 0.36 C, DEPENDING ON THE WEIGHT OF THE INDIVIDUAL UNIT.

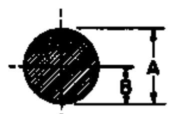
VOLUME OF INDIVIDUAL ARMOR UNIT = $0.16\ C^3$

WHERE: C = OVERALL DIMENSION OF UNIT
A = 0.20 C
B = 0.32 C
D = 0.057 C
ARMOR-LAYER THICKNESS (2 UNITS) = 1.020 C

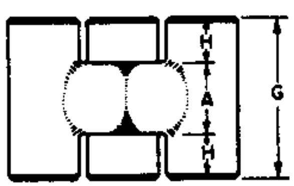
FIGURE 100
Dolos Specifications



PLAN



SECTION A-A



ELEVATION

	VOLUME OF INDIVIDUAL ARMOR UNITS (CU FT)										
	7.14	14.29	28.57	71.43	142.86	214.29	285.71	357.14	428.57	500.00	571.43
UNIT WEIGHT (LB/CU FT)	WEIGHT OF INDIVIDUAL ARMOR UNITS (TONS)										
140.0	0.50	1.00	2.00	5.00	10.00	15.00	20.00	25.00	30.00	35.00	40.00
149.5	0.53	1.07	2.14	5.34	10.68	16.02	21.36	26.70	32.04	37.38	42.71
156.0	0.56	1.11	2.23	5.57	11.14	16.71	22.29	27.86	33.43	39.00	44.57
162.0	0.58	1.16	2.31	5.79	11.57	17.36	23.14	28.93	34.71	40.50	46.29
	AVERAGE MEASURED THICKNESS OF ONE LAYER, UNIFORMLY PLACED (FT)										
	2.18	2.74	3.45	4.69	5.91	6.76	7.44	8.02	8.52	8.97	9.38
	AVERAGE MEASURED THICKNESS OF TWO LAYERS, PLACED PELLMELL (FT)										
	3.85	4.85	6.11	8.30	10.46	11.97	13.17	14.19	15.08	15.87	16.60
	NUMBER OF ARMOR UNITS PER 1,000 SQUARE FEET (ONE LAYER, UNIFORMLY PLACED)										
	161.34	101.63	64.02	34.80	21.86	16.71	13.83	11.92	10.60	9.52	8.74
	NUMBER OF ARMOR UNITS PER 1,000 SQUARE FEET (TWO LAYERS, PLACED PELLMELL)										
	247.85	156.12	98.35	53.45	33.58	25.67	21.25	18.31	16.28	14.63	13.43
SYMBOL	DIMENSIONS OF ARMOR UNITS (FT)										
A	1.05	1.32	1.66	2.25	2.84	3.25	3.58	3.85	4.09	4.31	4.51
B	0.52	0.66	0.83	1.13	1.42	1.62	1.79	1.93	2.05	2.15	2.25
C	1.25	1.58	1.99	2.70	3.41	3.90	4.29	4.62	4.91	5.17	5.41
D	1.78	2.24	2.82	3.83	4.82	5.52	6.08	6.55	6.96	7.33	7.66
E	1.09	1.37	1.72	2.34	2.95	3.38	3.72	4.00	4.25	4.48	4.68
F	3.22	4.06	5.11	6.93	8.74	10.00	11.01	11.86	12.60	13.26	13.87
G	2.09	2.63	3.32	4.51	5.68	6.50	7.15	7.70	8.19	8.62	9.01
H	0.52	0.66	0.83	1.13	1.42	1.62	1.79	1.93	2.05	2.15	2.25

NOTE: SHAPE AND DIMENSIONS OF UNIT BASED ON THOSE USED IN MODEL TESTS. AT PRESENT TIME, PATENTEE RECOMMENDS C = 1.25 A, AND FILLETS AT INTERSECTION OF HORIZONTAL AND VERTICAL MEMBERS WITH A RADIUS EQUAL TO A/4.

VOLUME OF INDIVIDUAL ARMOR UNIT = $6.48\ A^3$

WHERE:
A = diameter of leg
B = 0.5 A
C = 1.25 A
D = 1.75 A
E = 1.08 A
F = 3.16 A
G = 2 A
H = B = 0.5 A

ARMOR-LAYER THICKNESS (2 UNITS, RANDOM) = 3.68 A

FIGURE 101
Tribar Specifications

TABLE 11
Design Principles for Rubble-Mound Structures

Feature	Function
Armor units	Prevent their own dislodgement by wave forces; must be dense enough and heavy enough, or have sufficient interlocking ability, to prevent dislodgement.
Crest elevation	Sufficient crest elevation allows a structure to serve its function. In general, revetments should not be heavily overtopped. Breakwaters should provide adequate wave protection. Groins and jetties should have barriers that are impermeable to sand to a high enough elevation so that sand is prevented from leaking through or over the structure.
Underlayers	Prevent fines of smaller core or other underlayers from washing through voids in the structure. In the case of a revetment, the smallest underlayer prevents the in situ fines from washing through voids. (The smallest underlayer is sometimes referred to as a filter layer.)
Toe protection	Prevents undermining, by waves and currents, of a structure situated on unconsolidated material; allows for scour at the toe.
Core	Can render a breakwater or other rubble-mound structure impermeable due to the presence of the fine materials found in the relatively inexpensive quarry material used.
Flank protection	Protects the flanks of a revetment or jetty and prevents erosion from unraveling the structure.
Concrete cap	Stabilizes the crest of a low-height breakwater or other rubble-mound structure and provides an access and maintenance road on the breakwater.

TABLE 12

Suggested Stability Coefficient, K_D, Values for Use in Determining Armor-Unit Weight, W (No-Damage Criteria and Minor Overtopping)

Armor Units	[1]n	Placement	Slope (cot θ)	Structure Trunk [2]K_D		Structure Head K_D	
				Breaking Wave	Nonbreaking Wave	Breaking Wave	Nonbreaking Wave
Quarrystone Smooth, rounded...	2	Random	1.5 to 3.0	2.1	2.4	1.7	1.9
Smooth, rounded...	> 3	Random	[5]...	2.8	3.2	2.1	2.3
Rough, angular....	1	Random[3]	[5]...	[3]...	2.9	[3]...	2.3
Rough, angular....	2	Random	1.5 2.0 3.0	3.5	4.0	2.9 2.5 2.0	3.2 2.8 2.3
Rough, angular....	> 3	Random	[5]...	3.9	4.5	3.7	4.2
Rough, angular....	2	Special[4]	[5]...	4.8	5.5	3.5	4.5
Tetrapod and Quadripod.........	2	Random	1.5 2.0 3.0	7.2	8.3	5.9 5.5 4.0	6.6 6.1 4.4
Tribar.............	2	Random	1.5 2.0 3.0	9.0	10.4	8.3 7.8 7.0	9.0 8.5 7.7
Dolos..............	2	Random	[6]2.0 3.0	22.0	25.0	15.0 13.5	16.5 15.0
Modified Cube......	2	Random	[5]...	6.8	7.8	...	5.0
Hexapod............	2	Random	[5]...	8.2	9.5	5.0	7.0
Tribar.............	1	Uniform	[5]...	12.0	15.0	7.5	9.5
Quarrystone (K_{RR})[7] Graded, angular...	...	Random	...	2.2	2.5	...	...

[1]n is the number of units comprising the thickness of the armor layer.
[2]Applicable to slopes ranging from 1 on 1.5 to 1 on 5.
[3]The use of single layer of quarrystone armor units subject to breaking waves is not recommended, and only under special conditions for nonbreaking waves. When it is used, the stone should be carefully placed.
[4]Special placement with long axis of stone placed perpendicular to structure face.
[5]Until more information is available on the variation of K_D value with slope, the use of K_D should be limited to slopes ranging from 1 on 1.5 to 1 on 3. Some armor units tested on a structure head indicate a K_D-slope dependence.
[6]Stability of dolosse on slopes steeper than 1 on 2 should be investigated by site-specific model tests.
[7]For graded riprap, use K_{RR} instead of K_D.

(AFTER SHORE PROTECTION MANUAL, 1977)

(2) Armor units should be placed in at least a two-layer thickness except for uniformly fitted tribars and specially placed quarrystone. In these cases, single-layer placement is permissible if the structure is sited on a hard bottom, where foundation settlement or scour cannot occur, and if the contractor carefully keys and fits the units into place with the long axis perpendicular to the slope.

(3) Armor units for the structure head should be larger than those for the trunk and should extend along the trunk on the seaward and lee sides at least five wave heights from the end of the head. In lieu of larger armor units, the slope of the head section may be flattened sufficiently according to Equation (4-1).

(4) If the design condition is a nonbreaking wave at high tide, a check should be made to determine if the wave breaks at a lower tide. If this is the case, the appropriate K_D for breaking waves should be selected.

(5) Armor stones should be graded such that the maximum size is 1.25 and the minimum size is 0.75 W. Approximately 75 percent of the stones should be equal to, or larger than, W.

(6) Armor units should be hard, durable, and clean, without cracks, cleavages, or laminations. They should be chemically stable in fresh or salt water and should not weather due to freeze/thaw or wet/dry cycles.

(7) The smallest dimension of an armor unit should not be less than one-third of the largest dimension.

(8) Large concrete armor units should not be permitted to exhibit excessive movement during design conditions. A hydraulic model may be a beneficial design tool for large projects.

(9) Steel reinforcing can be used in concrete armor units to assist in handling and placing. Steel should be embedded at least 3 inches from all surfaces. The use of steel reinforcement for concrete armor units of less than 5 tons is rare. It is not mandatory to use reinforcing bars for larger units; however, reinforced units may have a longer life than nonreinforced units if they are to be subject to rocking.

(10) A graded riprap structure can be designed for waves with heights of less than about 5 feet by using the appropriate K_{RR} value in Table 12 (the value of K_{RR} is substituted for K_D in Equation (4-1). The value for the weight of the stone determined from Equation (4-1) for graded riprap is the 50-percent size in the gradation and is denoted W_{50}. The maximum stone size is 3.6 W_{50}, the minimum is 0.22 W_{50}, and 50 percent of the material weighs W_{50} or more.

(11) Armor units must extend over the crest onto the slope on the lee side. Lee-side armor units should weigh at least as much as front-slope units if the structure-crest freeboard above the design water level (F) is less than three-fourths of the design wave height. If the structure-crest freeboard is less than one-half of the design wave height, back-slope armor units may need to weigh more than those

on the front slope. If this be the case, model studies should be conducted. If the crest freeboard is greater than three-fourths of the design wave height, economy can be achieved by conducting a model study to optimize the armor-unit size for the back slope. Waves generated in the harbor may also govern back-slope armor design. (See Figure 102 for preliminary guidance on the relative weight of back-slope, WÚb¿, to front-slope, WÚf¿, armor units, where F = hÚs¿ - dÚs¿ = freeboard.)

(12) Armor-unit stabiLity should not depend on chinking.

(13) An allowance should be made for overbuilding the structure in the event settlement occurs.

(14) Allow at least a year for settlement prior to constructing a massive concrete cap on the breakwater.

3. CREST.

a. Crest Height. The height of the crest, hÚs¿, depends upon several factors, including crest and back-slope armor-unit stability and desired transmitted-wave height. Construction techniques must also be considered. Land-based equipment requires a working road out of the normal runup zone; floating plants require sufficiently deep and quiescent waters.

b. Crest Width. The crest width, B, should be sufficient to support construction and maintenance vehicles if the breakwater is to be constructed from land. The crest should be a minimum of three stones wide for stability. The minimum width for a working platform when land-based equipment is used is 12 to 15 feet. For smaller breakwater projects, when land-based construction equipment is used, economy can be achieved by building a temporary working platform or road on the core or underlayer material. The cap stones, or armor units, can then be placed with a minimum three-stone width, which can be less than 12 feet. This is done by starting the last layer on the cap and working landward. A disadvantage of this technique is that repairs require removal of stones or working from a floating plant. The minimum crest width, B, for a moderately overtopped breakwater is:

B = n kÚ[delta](W/wÚr¿)À1/3Ù (4-2)

WHERE: B = crest width

n = number of armor units comprising the crest width; a minimum of three

kÚdelta¿ = layer coefficient (see Table 13)

W = weight of individual armor unit in primary cover layer

WÚr¿ = unit weight of armor material (saturated surface dry)

[1] Table 13 presents data regarding the layer coefficient, kÚ[delta]¿, and porosity, P, for various armor units placed as specified in the number of layers of thickness specified. Values of kÚ[delta]¿ and/or P are required for calculating layer thickness, crest width, and the number of armor units in a breakwater.

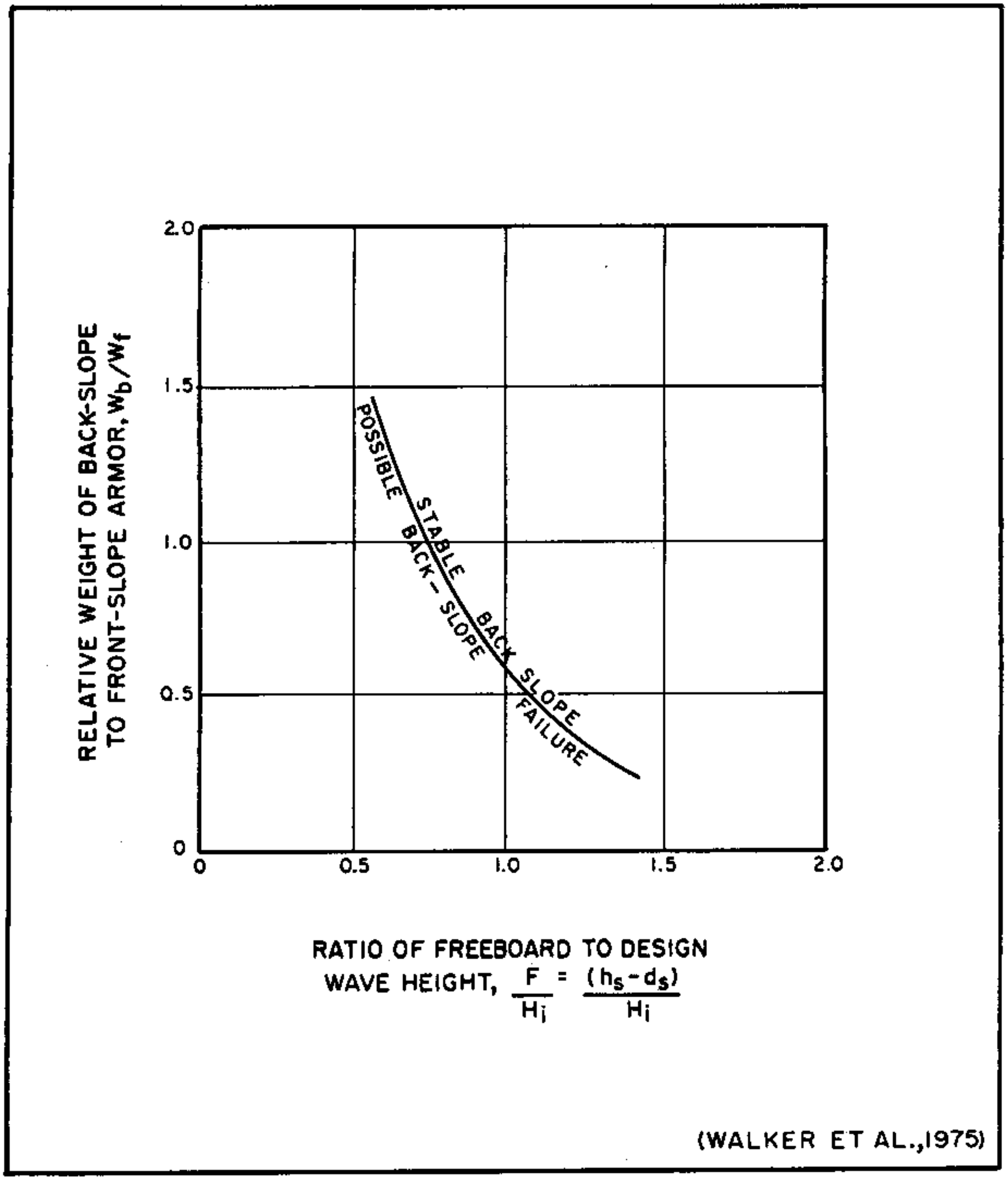


FIGURE 102
Back-Slope Stability

TABLE 13

Layer Coefficient and Porosity for Various Armor Units

Armor Units	Number Units Per Layer, n	Placement	Layer Coefficient, k_{Δ}	Porosity, P (percent)
Quarrystone (smooth).....	2	Random	1.02	38
Quarrystone (rough)......	2	Random	1.15	37
Quarrystone (rough)......	3	Random	1.10	40
Quarrystone..............	Graded	Random	....	37
Cube (modified)..........	2	Random	1.10	47
Tetrapod.................	2	Random	1.04	50
Quadripod................	2	Random	0.95	49
Hexapod..................	2	Random	1.15	47
Tribar...................	2	Random	1.02	54
Tribar...................	1	Uniform	1.13	47
Dolos....................	2	Random	1.00	63

(AFTER SHORE PROTECTION MANUAL, 1977)

Note: This table is used to find k_{Δ} for determining B and r.

4. LAYER THICKNESS. The layer thickness, r, for armor and underlayer units, measured perpendicular to the slope, is:

$$r = n\, k_{\Delta} \left(W/w_r\right)^{1/3} \qquad (4\text{-}3)$$

WHERE: r = layer thickness

n = number of units comprising the layer of interest; a minimum of two as a general rule (see Table 13 for exceptions)

k_{Δ} = layer coefficient (see Table 13)

W = weight of individual armor unit or stone in layer of interest

w_r = unit weight of armor material

5. NUMBER OF ARMOR UNITS. The placing density, or the number of armor or underlayer units per surface area per layer, N_r/A, is given by:

$$N_r/A = n\, k_{\Delta} \left[1 - (P/100)\right] \left(w_r/W\right)^{2/3} \qquad (4\text{-}4)$$

WHERE: N = number of individual units in layer of interest

A = surface area

n = number of units comprising the layer of interest; a minimum of two as a general rule (see Table 13 for exceptions)

k_{Δ} = layer coefficient (see Table 13)

P = Porosity (see Table 13)

wÚr¿ = unit weight of armor material (saturated surface dry)

W = weight of individual armor unit or stone in layer of interest

6. PRIMARY AND SECONDARY COVER LAYERS. For a breakwater subjected to nonbreaking waves, the primary armor unit should extend to a depth equal to 1.5 H below the minimum low©water level. Below this level, secondary armor units should be at least W/10 to W/15, where W is the weight of the individual armor unit in the primary cover layer. In cases where the breakwater is subject to breaking waves, the primary armor should extend to the bottom or to the bedding layer. Typical breakwater sections subjected to nonbreaking and breaking wave conditions are shown in Figures 103 and 104, respectively.

7. UNDERLAYERS. Underlayers provide a transition between armor units and the core. The number of underlayers depends upon the size of the armor units and the gradation of the core material. Underlayer stones should weigh W/10 to W/15, where W is the weight of the primary armor unit. A sufficient number of underlayers is required to ensure that core material is not washed through the voids of overlying stone. Table 14 gives recommended sizes and gradation of stone in a breakwater. This is for guidance only. The local quarries should be checked to ensure that stone of these gradations can be economically obtained. If not, size or gradation may have to be altered slightly or more distant quarries should be investigated. Woven plastic filter cloth may be used as an underlayer to retain fines. The filter cloth should not be overlain with stones larger than 1 ton. Sufficient folds should be allowed in the cloth to allow for settlement. Unwoven filter cloth should not be used as an underlayer.

TABLE 14
Guidance on Stone Size and Gradations for Breakwaters

Stone Size (weight)	Layer	Allowable Stone-Size Gradation (percent)
W.......................	Primary cover layer	125 to 75
W/10 to W/15............	Secondary cover layer and first underlayer	125 to 75
W/200 to W/4,000 or W/6,000................	Core and bedding layer	170 to 30

Note: gradation gives the allowable stone-size variation.

(AFTER SHORE PROTECTION MANUAL, 1977)

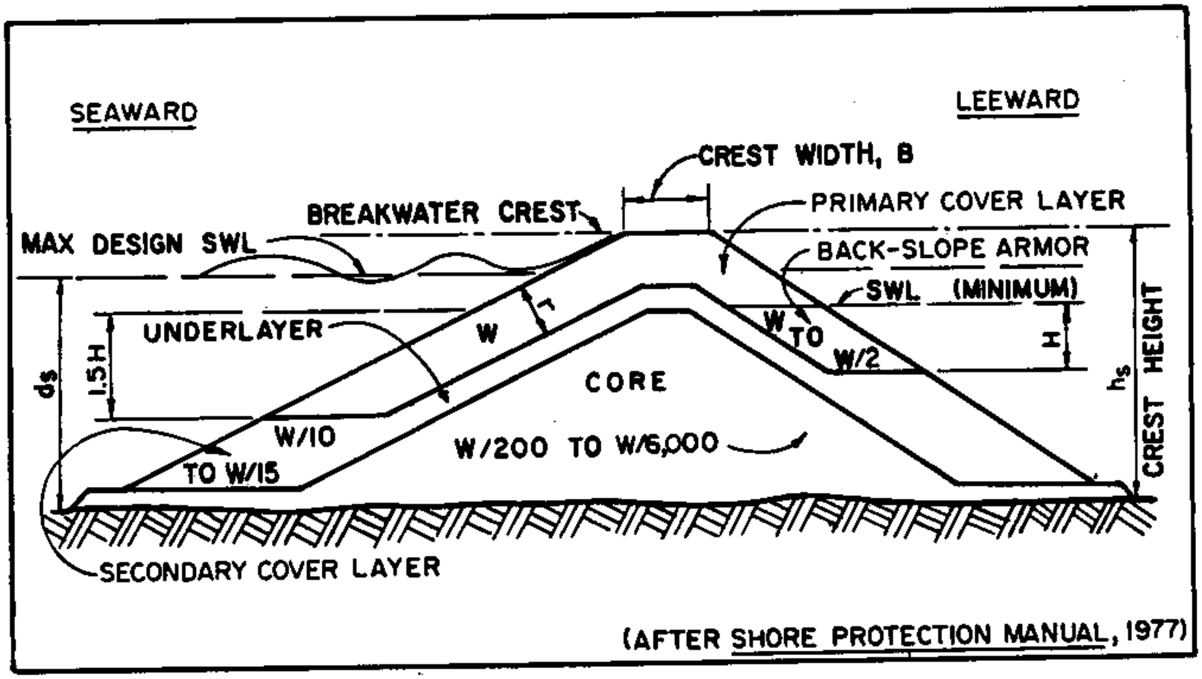


FIGURE 103
Typical Breakwater Section for Nonbreaking-Wave Conditions

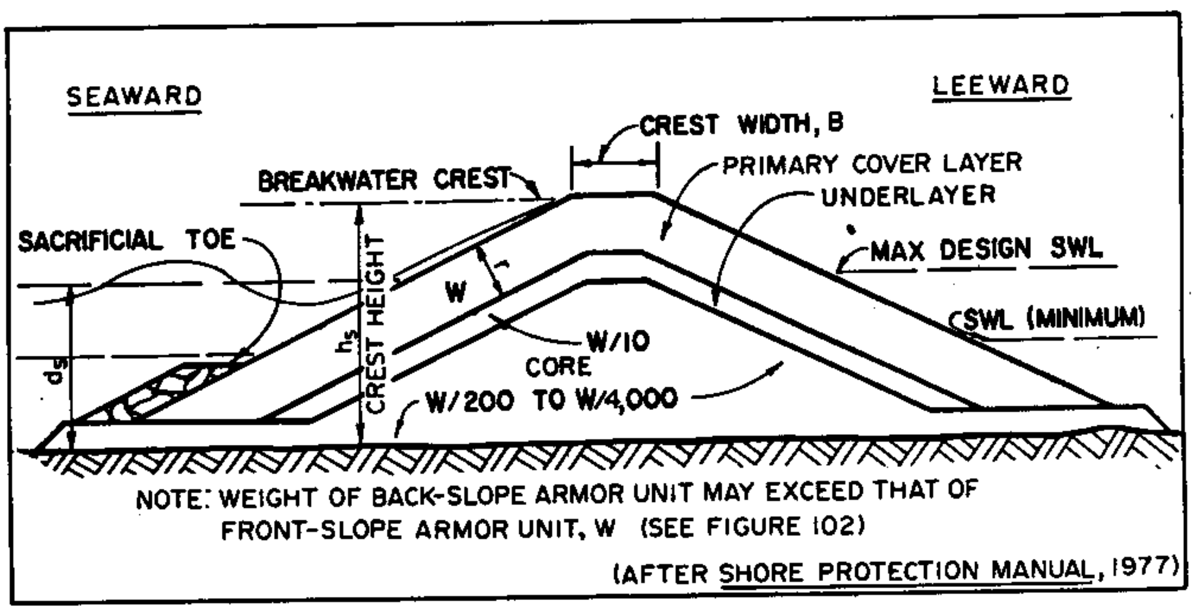


FIGURE 104
Typical Breakwater Section for Breaking-Wave Conditions

Wave Conditions and Typical Breakwater Section
for Breaking-Wave Conditions]

8. BEDDING LAYER. A bedding, or filter, layer is required over sandy bottoms to prevent the large armor stone from settling into the foundation material. The bedding should have at least a 2-foot thickness of quarry waste or quarry run. A layer of bedding material arranged in filter-like fashion, with the addition of plastic filter cloth, may be required over mud and clay bottoms to reduce displacement of the structure into thick deposits of soft mud. When the structure is built over a soft mud bottom, the stability of the foundation must be investigated for sliding and settlement. Gabions or foundation mattresses with reed matting protection or plastic filter cloth can be used to prevent stone material dropped from barges from settling deeply into the mud. Woven or unwoven filter cloth may also be used as a bedding layer over silty and sandy bottom.

9. SACRIFICIAL TOES. When scour is anticipated at the toe of a structure, toe protection is required. Scour often occurs when the structure is built in the surf zone. Effects of scour can be taken into account by placing armor or secondary stones on the bedding layer at the toe to act as a sacrificial toe. The toe should contain sufficient stone to protect the structure to the depth of anticipated scour.

10. CORE. The core material should be specified within sufficiently wide gradation limits such that a local quarry product can be used at a minimal cost. Dredged material, such as sand or coral, can be used as a core, provided adequate filters are installed to retain the fines.

EXAMPLE PROBLEM 27

Given:
a. Equivalent unrefracted deepwater wave height, H'_o = 10 feet
b. Bottom at -15 feet MLLW
c. Design tide range = -2 to + 7 feet MLLW (therefore, d_s ranges from 13 to 22 feet)
d. Wave period, T, ranges from 5 to 12 seconds.
e. Quarrystone unit weight, w = 160 pounds per cubic foot (available for land-based construction)
f. Quarrystone is rough, angular, and randomly placed.
g. Structure slope, cot [theta] = 1.5
h. The breakwater is fronted by a bottom slope, m = 0.033.

Find: Design a breakwater that will attenuate waves to 3.0 feet in its lee.

Solution: (1) Find design wave height at structure:

(a) Find H'_o/gT^2:

$$\frac{H'_o}{gT^2} = \frac{10}{(32.2)(12)^2} = 0.0022$$

(b) From Figure 42 for H'_o/gT^2 = 0.0022 and m = 0.033:

EXAMPLE PROBLEM 27 (Continued)

$$\frac{H_b}{H'_o} = 1.4$$

$$H_b = (1.4)(10) = 14 \text{ feet}$$

(2) Check breaker depth, d_b:

(a) Find H_b/gT^2:

$$\frac{H_b}{gT^2} = \frac{14}{(32.2)(12)^2} = 0.003$$

(b) From Figure 43 for $H_b/gT^2 = 0.003$ and $m = 0.033$:

$$\frac{d_b}{H_b} = 1.03$$

$$d_b = 1.03\, H_b$$

$$d_b = (1.03)(14) = 14.4 \text{ feet}$$

The structure will be subject to breaking waves when water level is lower than approximately 14.4 feet.

(3) Find d_s/H'_o for the lower and upper tide levels and H'_o/gT^2 for the wave-period range:

$$\frac{d_s}{H'_o} = \frac{13}{10} = 1.3 \qquad \text{(-2 feet MLLW tide)}$$

$$\frac{d_s}{H'_o} = \frac{22}{10} = 2.2 \qquad \text{(+7 feet MLLW tide)}$$

$$\frac{H'_o}{gT^2} = \frac{10}{(32.2)(12)^2} = 0.0124 \qquad \text{(5-second MLLW tide)}$$

$$\frac{H'_o}{gT^2} = \frac{10}{(32.2)(12)^2} = 0.00216 \qquad \text{(12-second MLLW tide)}$$

(4) Assuming a medium core height for a first approximation, refer to Subsection 3.4.b.(4)(b), Case 4: medium core height: $0.75 < h_c/d_s < 1.1$.; for $d_s/H'_o = 1.3$ to 2.2, use Figure 72 to find R/H'_o for cot [theta] = 1.5 and H'_o/gT^2 =

EXAMPLE PROBLEM 27 (Continued)

0.0124 (5-second period) or $H'_o/gT^2 = 0.00216$ (12-second period):

$$\frac{R}{H'_o} = 1.80 \qquad \text{(5-second period)}$$

$$\frac{R}{H'_o} = 2.40 \qquad \text{(12-second period)}$$

The maximum runup occurs for the 12-second-period wave.

(5) Using Equation (3-7), find runup, R, for a 12-second period wave:

$$R = (H'_o)(R/H'_o)(0.52)(1.06)$$

$$R = (10)(2.40)(0.52)(1.06) = 13.2 \text{ feet}$$

The maximum elevation of runup = 13.2 + 7 = +20.2 feet MLLW; this runup elevation occurs at high tide.

(6) Find weight of armor units, W, for front of structure (assuming salt water, w_w = 64 pounds per cubic foot):

(a) From Table 12 for breaking waves:

$K_D = 2.9$ for head and $K_D = 3.5$ for trunk

(b) Find W for armor units for structure head:

Using Equation (4-1), find W:

$$W = (w_r H^3)/[K_D (S_r - 1)^3 \cot \theta]$$

$$W = [(160)(14)^3]/\left[(2.9)\left(\frac{160}{64} - 1\right)^3(1.5)\right]$$

$$W = \frac{29{,}905 \text{ pounds}}{2{,}000 \text{ pounds per ton}}$$

W = 14.95 tons; use W = 15 tons

(c) Find W for armor units for structure trunk:

$$W = [(160)(14)^3]/\left[(3.5)\left(\frac{160}{64} - 1\right)^3(1.5)\right]$$

$$W = \frac{24{,}778 \text{ pounds}}{2{,}000 \text{ pounds per ton}}$$

W = 12.39 tons; use W = 12.5 tons

(7) Find crest height, hÚs¿, needed to attenuate waves to 3.0 feet:

EXAMPLE PROBLEM 27 (Continued)

Desired transmission coefficient, $K_t = \frac{H_t}{H_i} = \frac{3.0}{14} = 0.21$

$$\frac{H_i}{g\,T^2} = \frac{14}{(32.2)(12)^2} = 0.00302$$

From Figure 95 for $\frac{H_i}{g\,T^2} = 0.003$ and $K_t = 0.21$:

$$\frac{d_s}{h_s} \approx 0.7$$

$h_s \approx d_s/0.7 \approx 22/0.7 \approx 31.4$ feet; use $h_s = 31$ feet

Elevation of structure crest would be 31 - 15 = +16 feet MLLW.

In Step (5), above, the maximum elevation of runup was determined to be +20.2 feet MLLW. Thus, with the structure crest at +16 feet MLLW, the breakwater will be overtopped by the design wave by more than 4 feet. The stability of the back-slope armor must be checked.

(8) Check back-slope armor units for back-slope stability:

$$\frac{F}{H_i} = \frac{h_s - d_s}{H_i} = \frac{31 - 22}{14} = 0.64$$

From Figure 102 for $F/H_i = 0.64$ and front- and back-slope armor units of the same weight (that is, $W_b/W_f = 1.0$), there is a possibility of back-slope failure. To achieve back-slope stability, W_b/W_f should be increased to 1.23; this is done by increasing the size of the back-slope armor on the structure trunk:

$$\frac{W_b}{W_f} = 1.23 \text{ for back-slope stability}$$

$$W_b = 1.23\,W_f$$

$$W_b = (1.23)(12.5)$$

$W_b = 15.375$ tons; use $W_b = 15.5$ tons

(In a real design situation the economics of raising the structure height versus increasing the back-slope armor-unit weight should be investigated in order to minimize costs. This is normally done by conducting model tests.)

(9) Using Equation (4-2), find crest width, B:

EXAMPLE PROBLEM 27 (Continued)

$$B = n\,k_{\Delta}\,(W/w_r)^{1/3}$$

From Table 13 for rough quarrystone, randomly placed (and n = 3 from Subparagraph 3.3.b.):

$$k_{\Delta} = 1.10$$

$$B = (3)\,(1.10)\left[\frac{(12.5)(2{,}000)}{160}\right]^{1/3}$$

B = 17.77 feet; use B = 18 feet

(10) Using Equation (4-3), find front- and back-slope primary-cover layer thicknesses, r, assuming the layers are two stones thick:

$$r = n\,k_{\Delta}\,(W/w_r)^{1/3}$$

From Table 13 for rough quarrystone, randomly placed (n = 2):

$$k_{\Delta} = 1.15$$

(a) For the front slope:

$$r = (2)\,(1.15)\left[\frac{(12.5)\,(2{,}000)}{160}\right]^{1/3}$$

r = 12.39 feet; use r = 12.5 feet

(b) For the back slope:

$$r = (2)\,(1.15)\left[\frac{(15.5)(2{,}000)}{}\right]^{1/3}$$

r = 13.31 feet; use r = 13.5 feet

(11) Find weight of underlayer stone:

$$\frac{W}{10} = \frac{12.5}{10} = 1.25 \text{ tons}$$

(12) Using Equation (4-3), find underlayer thickness, r, for W = 1.5 tons, assuming the layer is two stones thick:

$$r = n\,k_{\Delta}\,(W/w_r)^{1/3}$$

$$r = (2)(1.15)\left[\frac{(1.25)\,(2{,}000)}{160}\right]^{1/3}$$

r = 5.75 feet; use r = 6 feet

(13) Using Equation (4-4), find number of primary armor units per surface area, NÚr¿/A, for the front slope:

EXAMPLE PROBLEM 27 (Continued)

$$N_r/A = n\, k_{\Delta}\, [1 - (P/100)](w_r/W)^{2/3}$$

From Table 13, $k_{\Delta} = 1.15$, $P = 37$

$$N_r/A = (2)(1.15)\left(1 - \frac{37}{100}\right)\left[\left(\frac{160}{(12.5)\,(2{,}000)}\right)\right]^{2/3}$$

$N_r/A = 0.0499$ per square foot

This means there are roughly 50 armor units per 1,000 square feet of cover layer surface area for the front slope. A similar calculation for the back slope can be done.

For breakwater cross section, see Figure 105.

Note: Calculated stone weights should be compared with those of local quarry products. Should minor deviation in weight and gradation result in significant cost reduction, these variations may be acceptable.

11. REVETMENTS. The same general principles used for design of breakwaters are used for design of revetments. The primary difference is that a revetment protects land from erosion. The revetment must have an adequate filter material to prevent fines in the in situ soils from washing through the voids of the structure. The filter can be either layers of stones or a woven plastic filter cloth. Allowance for scour at the toe should be given in developing the design wave for the revetment. The sides of revetments should extend sufficiently landward to prevent flank erosion (see DM-26.3, Subsection 2.3.b., Harbor Entrances on Open Coasts.)

EXAMPLE PROBLEM 28

Given:
- a. Equivalent unrefracted deepwater wave height, $H'_o = 10$ feet
- b. Bottom at -27 feet MLLW
- c. Design water level at +3 feet MLLW
- d. Wave period, $T = 8$ seconds
- e. Quarrystone unit weight, $w = 160$ pounds per cubic foot
- f. Quarrystone is rough, angular, and randomly placed.
- g. Structure slope, $\cot \theta = 2.0$
- h. The revetment is situated on a flat bottom.

Find: Design a quarrystone revetment.

Solution: (1) Find design wave height at structure:

$$L_o = (g/2\pi)\, T^2 = (32.2/2\pi)(8)^2 = 328 \text{ feet}$$

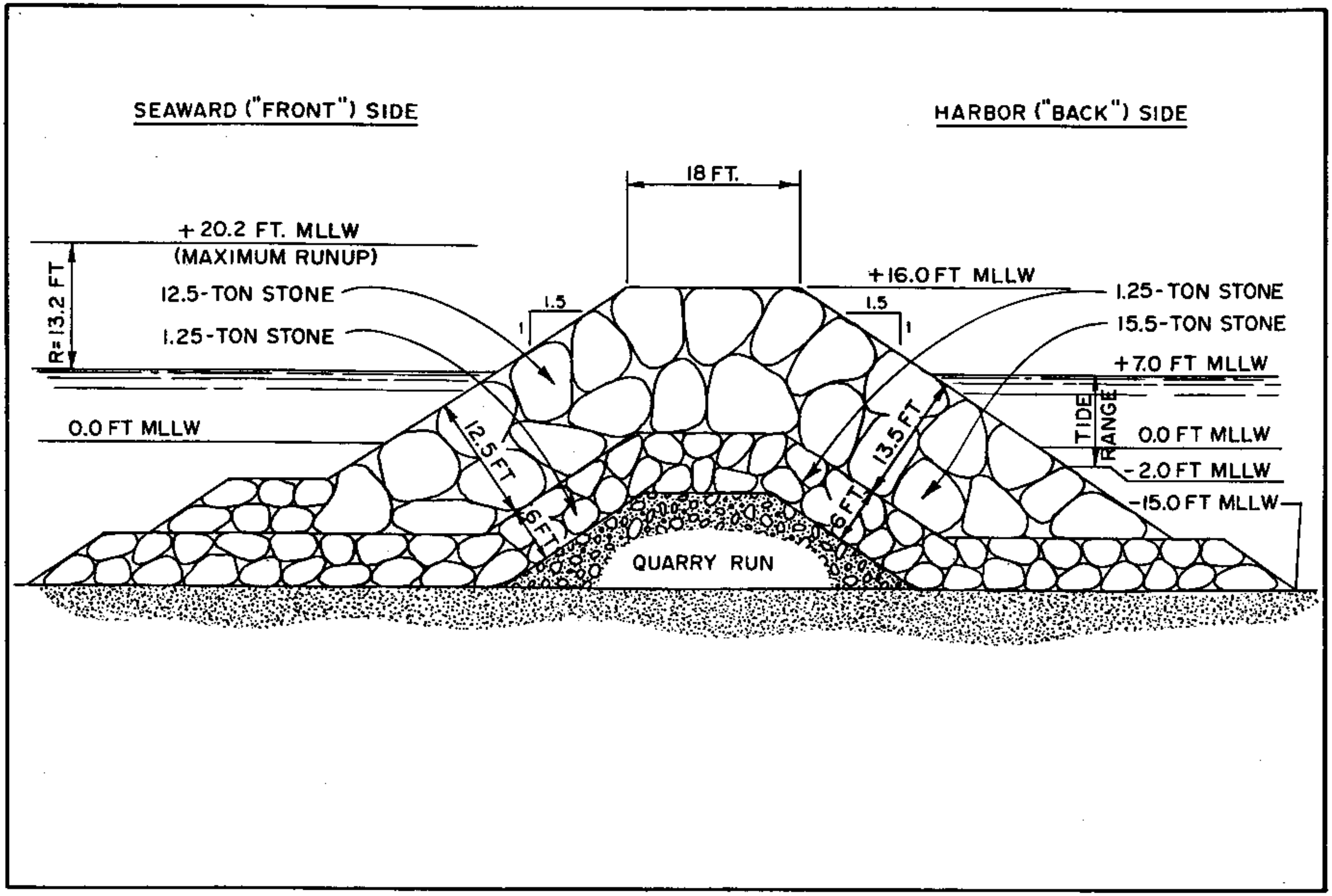


FIGURE 105
Solution to Example Problem 27

EXAMPLE PROBLEM 28 (Continued)

$d/L_o = 30/328 = 0.0915$

From Figure 2 for $d/L_o = 0.0915$:

$$\frac{H}{H'_o} = 0.94$$

$H = 0.94\ H'_o$

THEREFORE: $H = (0.94)(10) = 9.4$ feet

For a flat bottom, $H_b = 0.78\ d_b = (0.78)(30) = 23.4$ feet. Since $H < H_b$ (9.4 < 23.4), the revetment is subject to nonbreaking waves.

(2) Find d_s/H'_o and $H'_o/g\ T^2$:

$$\frac{d_s}{H'_o} = \frac{30}{10} = 3.0$$

$$\frac{H'_o}{g\ T^2} = \frac{10}{(32.2)(8)^2} = 0.00485$$

(3) Refer to Subsection 3.4.b.(2), Case 1: Embankment or Revetment, Quarrystone Armor; for $d_s/H'_o = 3.0$, use Figure 69 to find R/H'_o for $H'_o/g\ T^2 = 0.00485$ cot [theta] = 2.0):

$$\frac{R}{H'_o} = 1.25$$

(4) Using Equation (3-2), find runup, R:

$R = (H'_o)(R/H'_o)$

$R = (10)(1.25) = 12.5$ feet

For no overtopping, the elevation of structure should be at 12.5 + 3 = 15.5 feet MLLW.

(5) Find weight of armor units, W (assuming salt water, $w_w = 64$ pounds per cubic foot):

EXAMPLE PROBLEM 28 (Continued)

(a) From Table 12 for nonbreaking waves:

KÚD¿ = 4.0 for trunk

(b) Using Equation (4-1), find W for structure trunk:

W = (wÚr¿ HÀ3Ù)/[KÚD¿ (SÚr¿ - 1)À3Ù cot [theta]]

```
                        160
W = [(160)(9.4)À3Ù]/[(4)(ÄÄÄ - 1)À3Ù(2.0)]
                        64
```

```
        4,920 pounds
W = ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
    2,000 pounds per ton
```

W = 2.46 tons; use W = 2.5 tons

(6) Using Equation (4-3), find top layer thickness, r, assuming the layer is two stones thick:

r = n kÚ[delta]¿ (W/wÚr¿)À1/3Ù

From Table 13 for rough quarrystone, randomly placed (n = 2):

kÚ[delta]¿= 1.15

```
               (2.5)(2,000)
r = (2) (1.15) [ÄÄÄÄÄÄÄÄÄÄÄÄ]À1/3Ù
                    160
```

r = 7.24 feet; use r = 7.0 feet

(7) Find weight of underlayer stone:

```
W    2.5
ÄÄ = ÄÄÄ = 0.25 ton
10   10
```

(8) Using Equation (4-3), find underlayer thickness, r, for W = 0.25 ton, assuming the layer is two stones thick:

r = n kÚ[delta]¿ (W/wÚr¿) À1/3Ù

```
                 (0.25) (2,000)
r = (2) (1.15) [ ÄÄÄÄÄÄÄÄÄÄÄÄÄÄ]À1/3Ù
                      160
```

r = 3.363 feet; use r = 3.5 feet

For revetment cross section, see Figure 106.

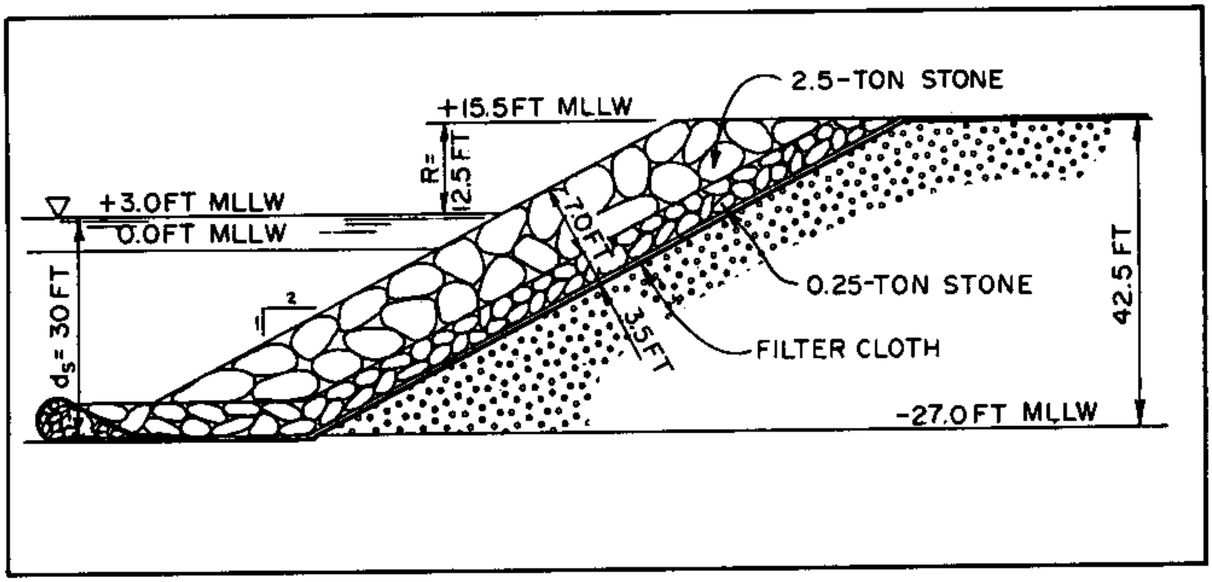


FIGURE 106
Solution to Example Problem 28

12. METRIC EQUIVALENCE CHART. The following metric equivalents were developed in accordance with ASTM E-621. These units are listed in the sequence in which they appear in the text of Section 4. Conversions are approximate.

3 inches = 7.6 centimeters
5 tons = 4,536 kilograms
5 feet = 1.5 meters
12 feet = 3.7 meters
15 feet = 4.6 meters
1 ton = 907 kilograms
2 feet = 61 centimeters

12. METRIC EQUIVALENCE CHART. The following metric equivalents were developed in accordance with ASTM E-621. These units are listed in the sequence in which they appear in the text of Section 4. Conversions are approximate.

3 inches = 7.6 centimeters
5 tons = 4,536 kilograms
5 feet = 1.5 meters
12 feet = 3.7 meters
15 feet = 4.6 meters
1 ton = 907 kilograms
2 feet = 61 centimeters

SECTION 5. WALL DESIGN

1. WAVE-INDUCED FORCES ON WALLS.

a. General. Procedures for calculating wave-induced forces on vertical walls are divided into three categories: nonbreaking waves, breaking waves, and broken waves. Procedures given below assume that wave action is only on the seaward side of the wall. In cases where wave action is also on the landward or harbor side of the wall, the calculation should assume a wave crest occurs on one side of the wall simultaneously with a wave trough on the opposite side of the wall. Calculations are also described for low-height walls, for walls built on a rubble base, for baffles, for wave attack at an angle other than normal to the wall, and for nonvertical walls.

b. Nonbreaking Waves. Waves impinging on a smooth, vertical-faced wall reflect most of their energy seaward. At the wall, a standing wave results for which the wave height at the wall is the sum of the incident and reflected wave heights. This standing wave is referred to as a clapotis. For perfect reflection, the resulting wave height is equal to twice the incident wave height.

Wave forces and moments for nonbreaking conditions can be estimated by the Miche-Rundgren method. It is important to realize that, when water is on both sides of the structure, the maximum force may be seaward in direction occurring when the wave trough is at the seaward side of the structure. The procedure for calculating nonbreaking wave forces is as follows (see Figure 107 for a definition of terms):

(1) Determine the wave height at the wall, H_w, by the equation:

$$H_w = 2\ H_i \tag{5-1}$$

WHERE: H = wave height at the wall
H_i = incident wave height

(2) Determine the increase in mean water level, h_o, above the still water level in Figure 85 as a function of H_i / g T^2 and H_i/d_s. If H_i/d_s is greater than 0.78, the wave is breaking or broken and procedures outlined in Subsection 5.1.c., Breaking Waves, or 5.1.d., Broken Waves, respectively, should be followed.

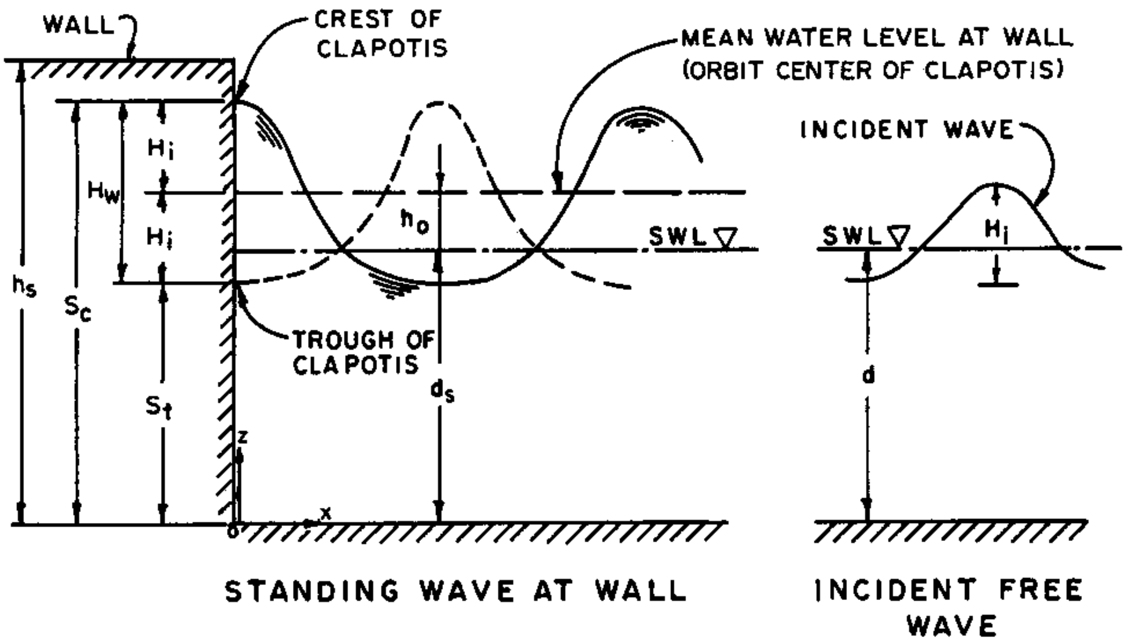
(3) The depth from the clapotis crest, S_c, and the depth from the clapotis trough, S_t, can be found by the following equations:

$$S_c = d_s + h_o + H_i \tag{5-2}$$

and

$$S_t = d_s + h_o - H_i \tag{5-3}$$

WHERE: S = depth from clapotis crest (S is measured along the z-coordinate axis, a vertical axis with its origin at the bottom)



WHERE: H_W = WAVE HEIGHT AT WALL

H_i = HEIGHT OF INCIDENT FREE WAVE (IN WATER OF DEPTH d)

h_o = HEIGHT OF CLAPOTIS ORBIT CENTER (MEAN WATER LEVEL AT WALL) ABOVE THE STILL WATER LEVEL (SWL)

d_s = DEPTH OF STRUCTURE FROM STILL WATER LEVEL

S_c = DEPTH FROM CLAPOTIS CREST $= d_s + h_o + H_i$

S_t = DEPTH FROM CLAPOTIS TROUGH $= d_s + h_o - H_i$

h_s = HEIGHT OF WALL

NOTE: a) CLAPOTIS MEANS STANDING WAVE

b) EQUATIONS PRESENTED IN SECTION 1 ARE BASED ON THE z-COORDINATE AXIS WITH ITS ORIGIN AT THE STILL WATER LEVEL. EQUATIONS IN SECTIONS 5 AND 7 ARE BASED ON THE z-COORDINATE AXIS WITH ITS ORIGIN AT THE BOTTOM.

(AFTER SHORE PROTECTION MANUAL, 1977)

FIGURE 107
Definition of Terms for Wave-Induced Forces on Walls
(Nonbreaking Waves)

d_s = depth at structure from still water level

h_o = height of clapotis orbit center (mean water level) above SWL

H_i = incident wave height

S_t = depth from clapotis trough

(4) Wave-induced pressure distribution for nonbreaking waves is shown schematically in Figure 108. The pressure when the clapotis crest is at the wall is p_c, and the pressure when the clapotis trough is at the wall is p_t. Equations (5-4) through (5-7) give the values of p_c and p_t at different values of z. When the clapotis crest is at the wall (Figure 108A), pressure increases from zero at the free water surface (Equation (5-4)) to $w_w d_s + p_1$ at the bottom (Equation (5-5)). When the clapotis trough is at the wall (Figure 108B), pressure increases from zero at the free water surface (Equation (5-6)) to $w_w d_s - p_1$ at the bottom (Equation (5-7)).

$$p_c = 0 \quad \text{at } z = S_c = d_s + h_o + H_i \tag{5-4}$$

$$p_c = w_w d_s + P_1 \quad \text{at } z = 0 \tag{5-5}$$

$$p_t = 0 \quad \text{at } z = S_t = d_s + h_o + H_i \tag{5-6}$$

$$p_t = w_w d_s - P_1 \quad \text{at } z - 0 \tag{5-7}$$

WHERE: p_c = pressure when clapotis crest is at wall

z = vertical distance along a coordinate axis with its origin at the bottom

S_c = depth from clapotis crest

w_w = unit weight of water

d_s = depth at structure from still water level

$P_1 = w_w H_i / \cosh(2 \pi d_s / L)$ = nonbreaking-wave pressure difference from still water hydrostatic pressure as clapotis crest (trough) passes (5-8)

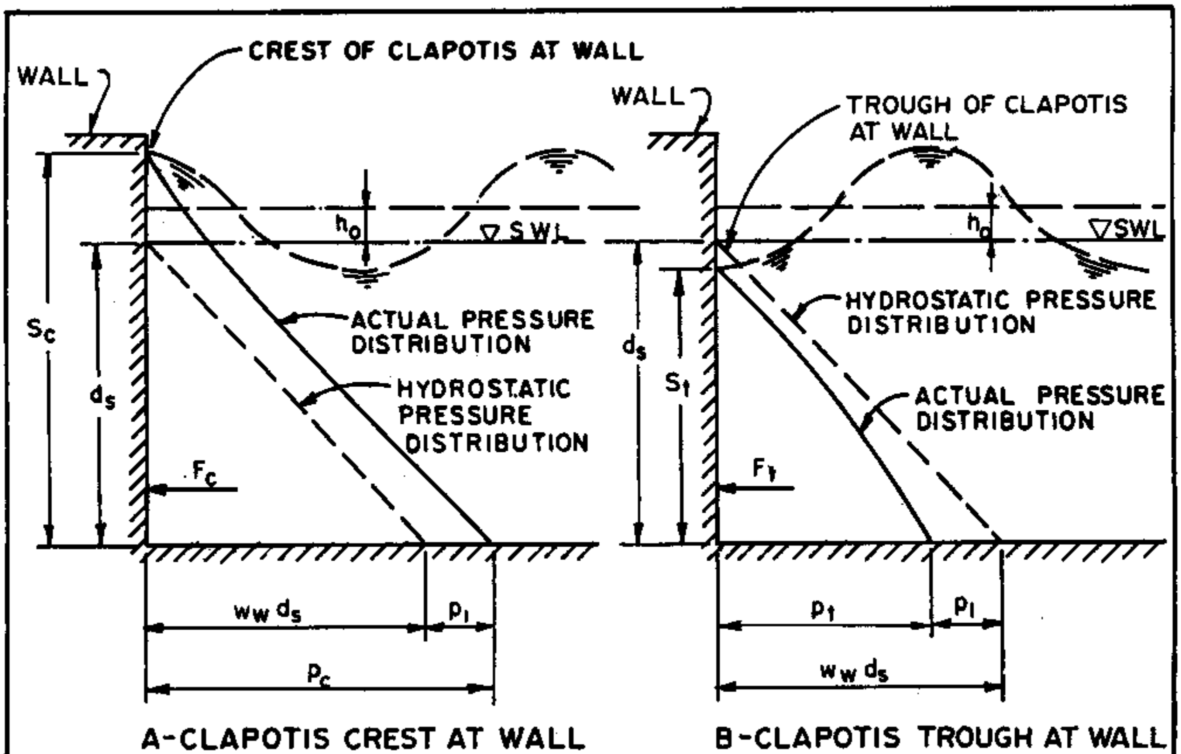
H_i = incident wave height

L = wavelength

p_t = pressure when clapotis trough is at wall

S_t = depth from clapotis trough

As a first approximation, the pressure distribution can be assumed to be a straight line between p at the crest or trough and p at the bottom. (See Figure 108.)



WHERE: w_w = UNIT WEIGHT OF WATER

p_1 = NONBREAKING-WAVE PRESSURE DIFFERENCE FROM STILL WATER HYDROSTATIC PRESSURE AS CLAPOTIS CREST (OR TROUGH) PASSES

p_c (or p_t) = PRESSURE WHEN CREST (OR TROUGH) IS AT WALL

F_c (or F_t) = FORCE WHEN CREST (OR TROUGH) IS AT WALL

(AFTER SHORE PROTECTION MANUAL, 1977)

FIGURE 108
Wave-Induced Pressure Distribution for Nonbreaking Waves at a Wall

Waves at a Wall]

The wave-induced forces and moments associated with the above pressures may be determined by assuming a straight-line pressure distribution between the water surface and the bottom and then using the following equations:

$$F_c \left(\frac{1}{2}\right) (w_w d_s + p_1)(d_s + h_o + H_i) \tag{5-9}$$

$$M_c \left(\frac{1}{6}\right) (w_w d_s + p_1)(d_s + h_o + H_i)^2 \tag{5-10}$$

$$F_t \left(\frac{1}{2}\right) (w_w d_s \; p_1)(d_s + h_o - H_i) \tag{5-11}$$

$$M_t = \left(\frac{1}{6}\right) (w_w d_s - p_1) (d_s + h_o - H_i)^2 \tag{5-12}$$

WHERE: F_c = Nonbreaking-wave force, per unit length of wall, when the crest of the clapotis is at the wall

M_c = Nonbreaking-wave moment, per unit length of wall, when the crest of the clapotis is at the wall

F_t = Nonbreaking-wave force, per unit length of wall, when the trough of the clapotis is at the wall

M_t = Nonbreaking-wave moment, per unit length of wall, when the trough of the clapotis is at the wall

(5) A more accurate determination of the integrated forces, F, and moments (about the mudline), M, for a nonovertopped wall subjected to nonbreaking waves can be obtained using Figures 109 and 110, respectively. On each figure, the values of $H_i/g T^2$ and H_i/d_s are used to determine the dimensionless force, $F/(w_w d_s^2)$, and the dimensionless moment, $M/(w_w d_s^3)$, respectively. The method for determining F and M is explained below.

On each of Figures 109 and 110, the upper family of curves is used to determine the dimensionless force or dimensionless moment, respectively, when the crest is at the wall; the lower family of curves is used to determine the force or moment when the trough is at the wall. On each of Figures 109 and 110, the horizontal line separating the two families of curves represents the dimensionless horizontal hydrostatic force, $(1/2)(w_w d_s^2)$, and moment, $(1/6)(w_w d_s^3$, respectively, exerted on a wall in still water. Once $F/w_w d_s)$ and $M/(w_w d_s^3)$ have been determined for the wave crest or trough from the appropriate figure, then the wave-induced horizontal force and moment acting on the wall may be determined using the following equations:

$$F_{c \text{ or } t} = \left(\frac{F}{w_w d_s^2}\right)_{c \text{ or } t} (w_w d_s^2) \tag{5-13}$$

```
                                M
MÚc¿ or  t = (ÄÄÄÄÄÄÄÄÄÄÄÄÄÄ)          (wÚw¿ dÚs¿À2Ù)              (5-14)
              wÚw¿ dÚs¿À2Ù  c or t
```

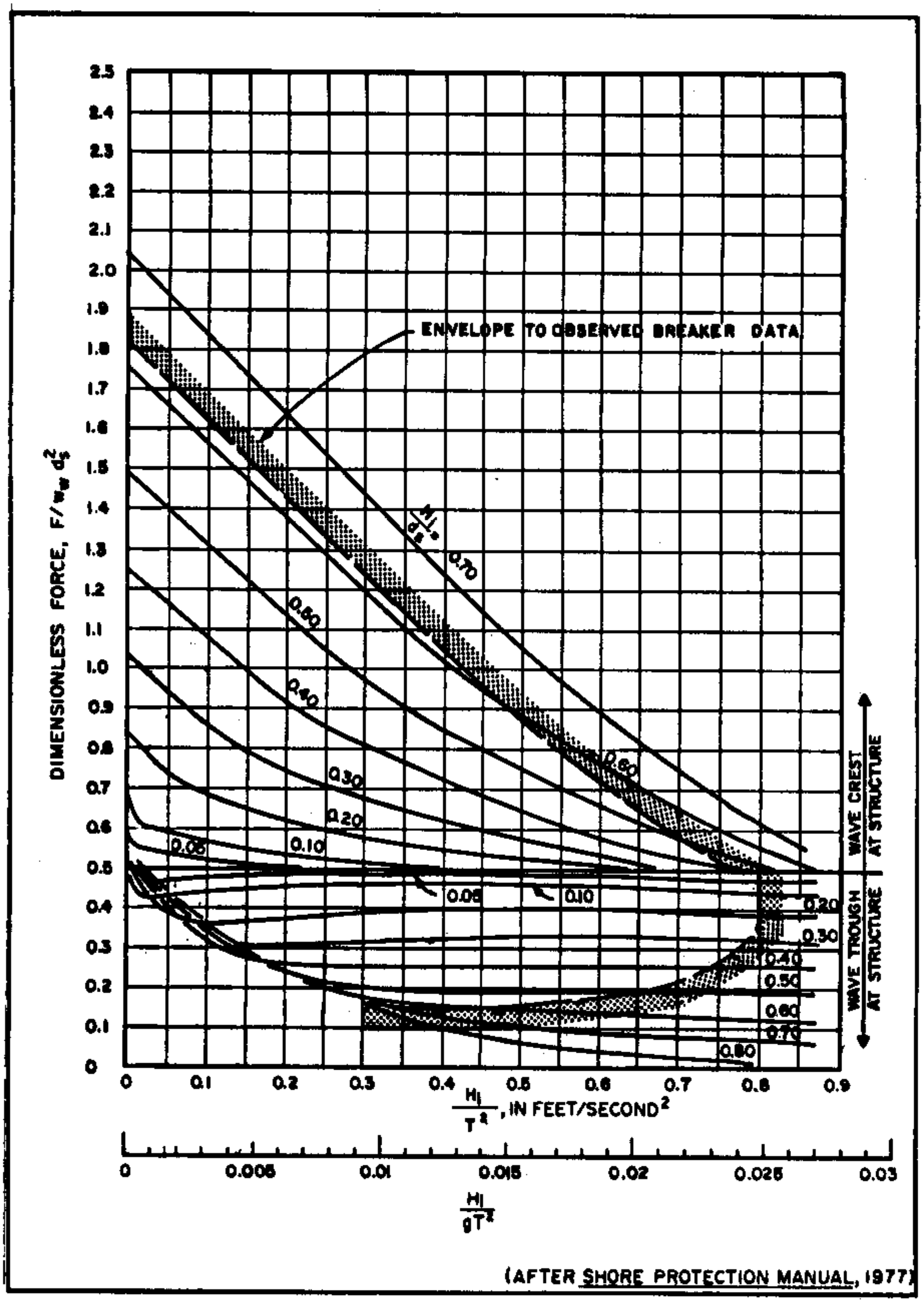


FIGURE 109
Nonbreaking-Wave Force

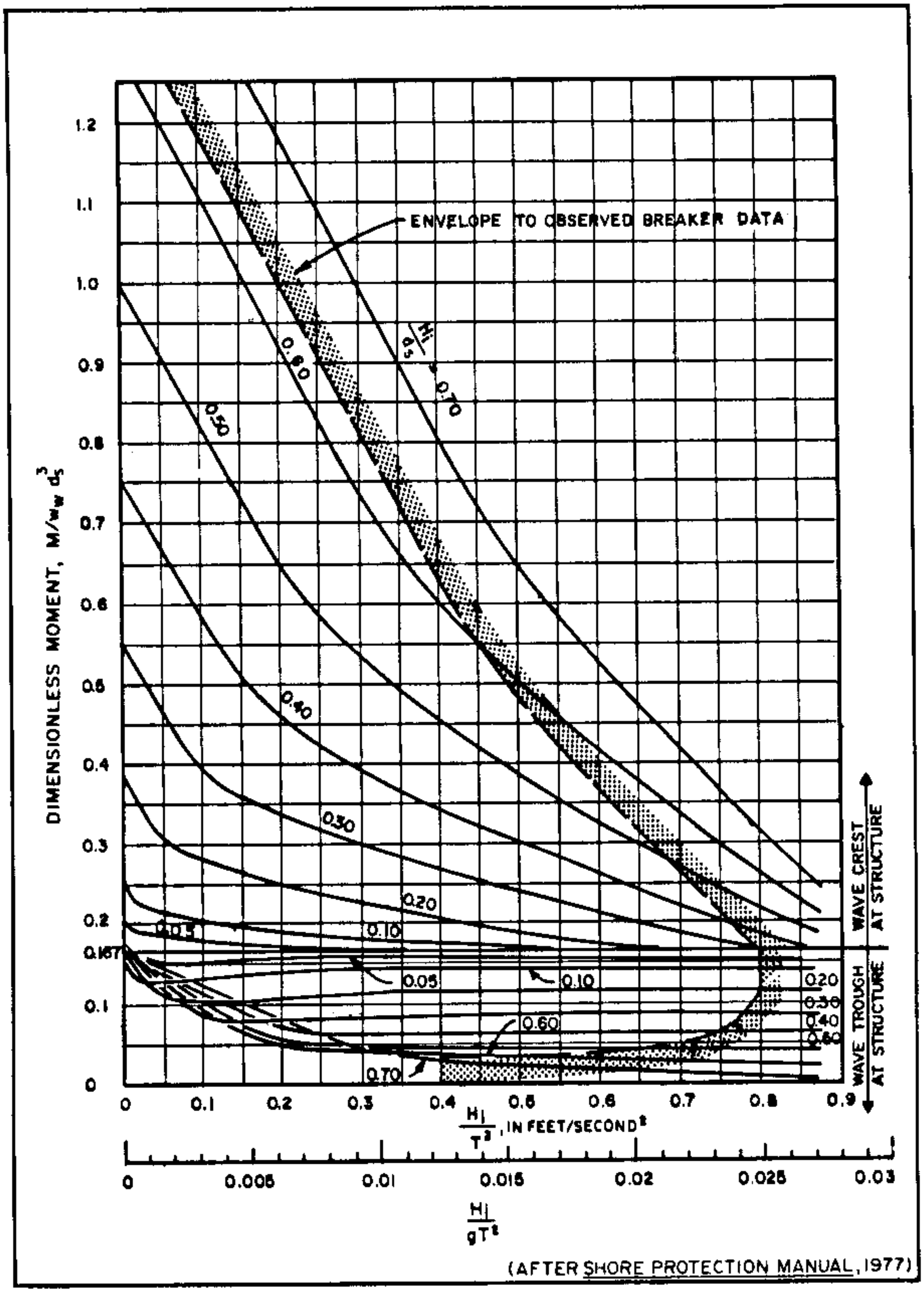


FIGURE 110
Nonbreaking-Wave Moment

If there is water on the lee side of the wall and there is no wave action present on the lee side, then the net force, F_{net}, acting on the wall will be the difference between the wave-induced force on the seaward side of the wall (at either the crest or trough) and the hydrostatic force on the lee side of the wall. Similarly, the net moment acting on the wall will be the difference between the wave-induced moment on the seaward side on the wall and the hydrostatic moment on the lee side of the wall.

Where wave action is present on the lee side of the wall, then the maximum net force and moment are determined by assuming a wave crest occurs on one side of the wall simultaneously with a wave trough on the opposite side of the wall. This situation arises when waves are transmitted over or through the wall or when waves from another source, such as reflected waves or locally generated waves, impinge upon the wall.

EXAMPLE PROBLEM 29

Given:
a. Incident wave height, H = 10 feet
b. Water depth, d = 20 feet
c. Wave period, T = 8 seconds
d. Sheet-pile wall as shown in Figure 111A; h_s = 40 feet

Find: The net force and moment on the sheet-pile wall.

Solution: (1) Find L at the structure depth:

$$L_o = (g/2\pi)\,T^2 = (32.2/2\pi)(8)^2 = 328 \text{ feet}$$

$$\frac{d_s}{L_o} = \frac{20}{328.0} = 0.061$$

From Figure 2 for d/L_o = 0.061:

$$\frac{d_s}{L} = 0.105$$

$$L = \frac{d_s}{0.105} = \frac{20}{0.105} = 190 \text{ feet}$$

(2) Determine $H_i/g\,T^2$ and H_i/d_s:

$$\frac{H_i}{g\,T^2} = \frac{10}{(32.2)(8)^2} = 0.00485$$

$$\frac{H_i}{d_s} = \frac{10}{20} = 0.5$$

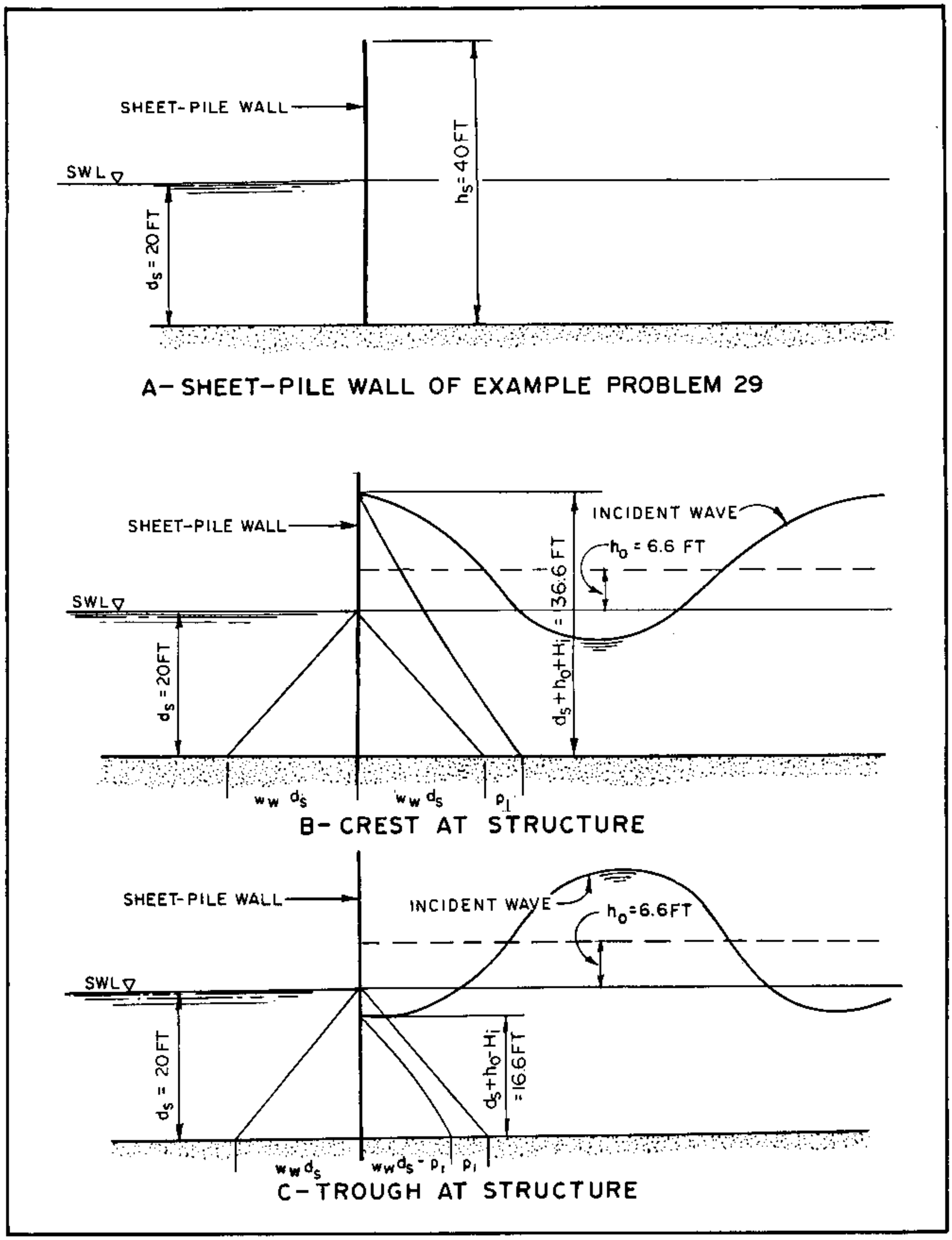


FIGURE 111
Diagrams for Example Problem 29

EXAMPLE PROBLEM 29 (Continued)

(3) Find h_o:

From Figure 85 for $H_i/g T^2 = 0.00485$ and $H_i/d_s = 0.5$:

$$\frac{h_o}{H_i} = 0.66$$

$$h_o = 0.66\ H_i$$

$$h_o = (0.66)(10) = 6.6 \text{ feet}$$

(4) Using Equations (5-2) and (5-3), respectively, find S_c and S_t:

$$S_c = d_s + h_o + H_i = 20 + 6.6 + 10$$

$$S_c = 36.6 \text{ feet}$$

$$S_t = d_s + h_o - H_i = 20 + 6.6 - 10$$

$$S_t = 16.6 \text{ feet}$$

(5) Using Equation (5-8), find p_1:

$$p_1 = \frac{w_w\ H_i}{\cosh(2\pi\ d_s/L)}$$

Find $\cosh(2\pi\ d_s/L)$:

$$\frac{2\pi\ d_s}{L}\ \ \frac{2\pi\ (20)}{190} = 0.661; \text{ use } 0.66$$

From Figure 3, $\cosh(0.66) = 1.23$

$$p_1 = \frac{(64)(10)}{1.23} = 520.3 \text{ pounds per square foot}$$

(6) Find net force and moment when wave crest is at structure (see Figure 111B):

(a) As a first approximation, determine net force, F_{net}, and moment, M_{net}, on structure using pressure at bottom, p_c, and assume a straight-line pressure distribution up to the water surface, where $p_c = 0$.

Note: Water level on leeward side will be taken as 20 feet; it is assumed there is no wave transmission and no wave action present on the leeward side.

EXAMPLE PROBLEM 29 (Continued)

$$F_{net} = F_c - \frac{1}{2} w_w d_s^2$$

Using Equation (5-9) and the above equation:

$$F_{net} = \left(\frac{1}{2}\right) (w_w d_s + p_1) (d_s + h_o + H_i) - \frac{1}{2} w_w d_s^2$$

(Assuming salt water, w_w = 64 pounds per cubic foot.)

$$F_{net} = \left(\frac{1}{2}\right) [(64) (20) + 520.3] (20 + 6.6 + 10) - \left(\frac{1}{2}\right) [(64) (20)^2]$$

F_{net} = 32,945 - 12,800

F_{net} = 20,145 pounds per foot (force with crest at structure)

$$M_{net} = M_c - \frac{1}{6} w_w d_s^3$$

Using Equation (5-10) and the above equation:

$$M_{net} = \left(\frac{1}{6}\right) (w_w d_s + p_1) (d_s + h_o + H_i)^2 - \frac{1}{6} w_w d_s^3$$

$$M_{net} = \left(\frac{1}{6}\right) [(64)(20) + 520.3] [(36.6)^2] - \left(\frac{1}{6}\right) [(64) (20)^3]$$

M_{net} = 401,935 - 85,333

M_{net} = 316,602 foot-pounds per foot (moment with crest net at structure)

(b) Now use Figures 109 and 110, respectively, to determine net wave force and moment when crest is at structure:

From Figure 109 for $H_i/g T^2$ = 0.00485 and H_i/d_s = 0.5 (wave crest at structure):

$$\frac{F_c}{w_w d_s^2} = 1.21$$

Using Equation (5-13):

```
          ÚÄ            Ä¿
          ³    FÚc¿      ³
FÚc¿  =   ³ ÄÄÄÄÄÄÄÄÄÄÄÄ ³  (wÚw¿ dÚs¿À2Ù)
          ³ wÚw¿ dÚs¿À2Ù ³
          ÀÄ            ÄÙ
```

EXAMPLE PROBLEM 29 (Continued)

$$F_c = (1.21)\,[(64)\,(20)^2]$$

$$F_c = 30{,}976 \text{ pounds per foot}$$

$$F_{net} = F_c - \frac{1}{2}\,d_s^2$$

$$F_{net} = 30{,}976 - \left(\frac{1}{2}\right)[(64)\,(20)^2]$$

$$F_{net} = 30{,}976 - 12{,}800$$

$$F_{net} = 18{,}176 \text{ pounds per foot (force with crest at structure)}$$

From Figure 110 for $H_i/g\,T^2 = 0.00485$ and $H_i/d_s = 0.5$ (wave crest at structure):

$$\frac{M_c}{w\,d_s^3} = 0.72$$

Using Equation (5-14):

$$M_c = \left(\frac{M_c}{w\,d_s^3}\right)(w\,d_s^3)$$

$$M_c = (0.72)\,[(64)\,(20)^3]$$

$$M_c = 368{,}640 \text{ foot-pounds per foot}$$

$$M_{net} = M_c - \frac{1}{6}\,w\,d_s^3$$

$$M_{net} = 368{,}640 - \left(\frac{1}{6}\right)[(64)\,(20)^3]$$

$$M_{net} = 368{,}640 - 85{,}333$$

$$M_{net} = 283{,}307 \text{ foot-pounds per foot (moment with crest at structure)}$$

Note: Values for F and M, for the wave crest at the structure, determined in step (b) (using Figures 109 and 110) are slightly lower than those calculated in step (a). This discrepancy is due to the assumption in step (a) that the pressure distribution is a straight line.

(7) Find force and moment when wave trough is at structure (see Figure 111C).

EXAMPLE PROBLEM 29 (Continued)

(a) As a first approximation, assume straight-line pressure distribution:

$$F_{net} = F_t - \frac{1}{2} w_w d_s^2$$

Using Equation (5-11) and the above equation:

$$F_{net} = \left(\frac{1}{2}\right)(w_w d_s - p_1)(d_s + h_o - H_i) - \frac{1}{2} w_w d_s^2$$

$$F_{net} = \left(\frac{1}{2}\right) [(64)(20) - 520.3] (20 + 6.6 - 10) - \left(\frac{1}{2}\right) [(64)(20)^2]$$

$$F_{net} = 6{,}306 - 12{,}800$$

F_{net} = -6,494 pounds per foot (force with trough at structure)

$$M_{net} = M_t - \frac{1}{6} w_w d_s^3$$

Using Equation (5-12) and the above equation:

$$M_{net} = \left(\frac{1}{6}\right)(w_w d_s - p_1)(d_s + h_o - H_i)^2 - \frac{1}{6} w_w d_s^3$$

$$M_{net} = \left(\frac{1}{6}\right) [(64)(20) - 520.3] (20 + 6.6 - 10)^2 - \left(\frac{1}{6}\right) [(64)(20)^3]$$

$$M_{net} = 34{,}890 - 85{,}333$$

M_{net} = -50,443 foot-pounds per foot (moment with trough at structure)

(b) Now use Figures 109 and 110, respectively, to determine wave force and moment (when trough is at structure):

From Figure 109 for $H_i/g T^2 = 0.00485$ and $H_i/d_s = 0.5$ (wave trough at structure):

$$\frac{F_t}{\qquad} = 0.28$$

```
wÚw¿ dÚs¿À2Ù

Using Equation (5-13):
       ÚÄ             Ä¿
       ³    FÚt¿       ³
FÚt¿ = ³ ÄÄÄÄÄÄÄÄÄÄÄÄ ³ (wÚw¿ dÚs¿À2Ù)
       ³ wÚw¿ dÚs¿À2Ù ³
       ÀÄ             ÄÙ
```

EXAMPLE PROBLEM 29 (Continued)

$$F_t = (0.28)\ [(64)(20)^2]$$

$$F_t = 7{,}168 \text{ pounds per foot}$$

$$F_{net} = F_t - \frac{1}{2} w_w d_s^2$$

$$F_{net} = 7{,}168 - \left(\frac{1}{2}\right) (64)(20)^2$$

$$F_{net} = 7{,}168 - 12{,}800$$

$$F_{net} = -5{,}632 \text{ pounds per foot (force with trough at structure)}$$

From Figure 110 for $H_i/g T^2 = 0.00485$ and $H_i/d_s = 0.5$ (wave trough at structure):

$$\frac{M_t}{w_w d_s^3} = 0.06$$

Using Equation (5-14):

$$M_t = \left(\frac{M_t}{w_w d_s^3}\right)(w_w d_s^3)$$

$$M_t = (0.06)\ [(64)\ (20)^3]$$

$$M_t = 30{,}720 \text{ foot-pounds per foot}$$

$$M_{net} = M_t - \frac{1}{6} w_w d_s^3$$

$$M_{net} = 30{,}720 - \left(\frac{1}{6}\right) w_w d_s^3$$

$$M_{net} = 30{,}720 - 85{,}333$$

$$M_{net} = -54{,}613 \text{ foot-pounds per foot (moment with trough at structure)}$$

Therefore, the wave crest at the structure provides maximum net forces and moments and these should be used for structural design.

(6) Forces on segments of the wall can be estimated by calculating the area in the idealized pressure distribution suggested in item (4) under Subsection 5.b., Nonbreaking Waves.

and the moment is:

$$M = \int_{z_1}^{z_2} z \quad p \; dz \tag{5-16}$$

WHERE: F = force

z_1 = depth from point 1

z_2 = depth from point 2

p = pressure

M = moment

z = vertical distance along a coordinate axis with its origin at the bottom

This procedure can be used to estimate forces on walls that are overtopped and/or do not extend to the bottom.

(7) Walls of low height may be overtopped and the pressure distribution is truncated as shown in Figure 112. Forces are reduced when the wave overtops the structure; the reduced force is calculated by applying a correction factor, termed a reduction factor, to the force as calculated above. No correction is necessary for analyzing the case where the wave trough is at the structure unless the elevation of the trough is higher than the wall. Reduced forces, F', and moments (about the mudline), M', are given by:

F' = rÚf¿ F (5-17)

and

M' = rÚm¿ M (5-18)

WHERE: F' = reduced force for overtopped wall

rÚf¿ = reduction factor for force

F = force determined for nonovertopped wall

M' = reduced moment for overtopped wall

rÚm¿ = reduction factor for moment

M = moment determined for nonovertopped wall

Reduction factors, rÚf¿ and rÚm¿ , are determined from Equations (5-19) and (5-20), respectively:

$$r_f = \left(\frac{h_s}{S}\right)\left(2 - \frac{h_s}{S}\right) \qquad \text{when } \frac{h_s}{S} < 1.0 \tag{5-19}$$

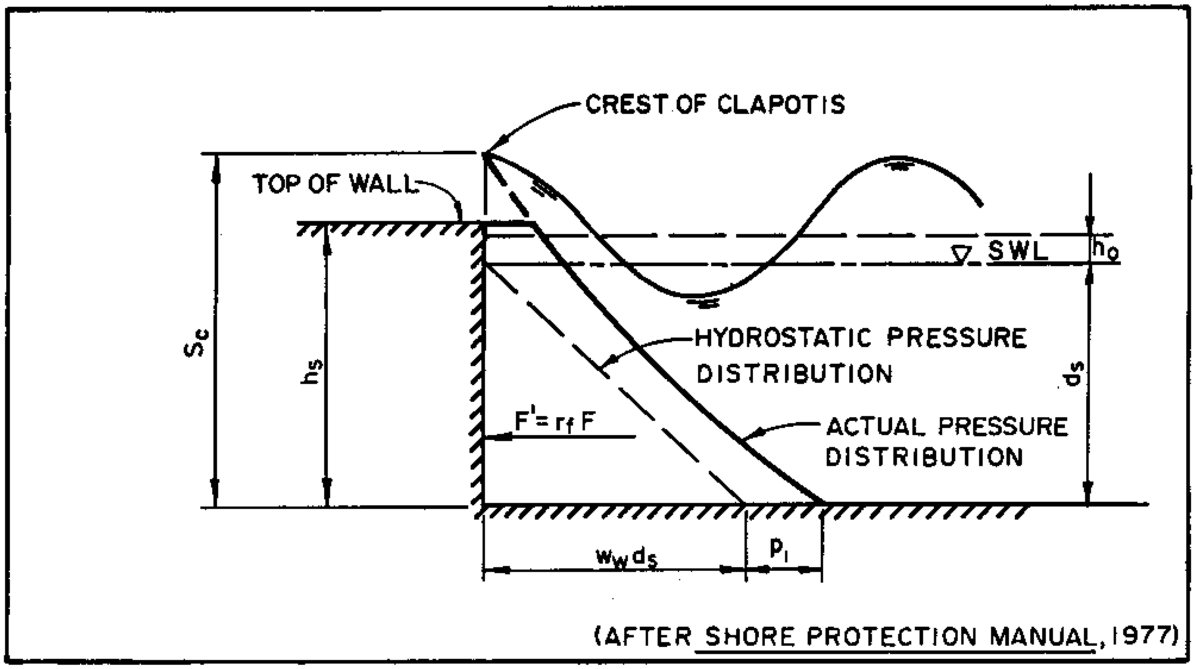


FIGURE 112
Pressure Distribution for Overtopped Low-Height Wall

or

$$r_f = 1.0 \qquad \text{when } \frac{h_s}{S} >/= 1.0$$

and

$$r_m = \left(\frac{h_s}{S}\right)^2 \left[3 - 2\left(\frac{h_s}{S}\right)\right] \qquad \text{when } \frac{h_s}{S} < 1.0 \qquad (5\text{-}20)$$

$$r_m = 1.0 \qquad \text{when } \frac{h_s}{S} >/= 1.0$$

WHERE: r_f = reduction factor for force
h = height of wall
S = depth from crest, S_c, or trough, S_t, of wave
r_m = reduction factor for moment

EXAMPLE PROBLEM 30

Given: a. Incident wave height, H_i = 10 feet
b. Water depth, d = 20 feet
c. Wave period, T = 6 seconds
d. Sheet-pile wall as shown in Figure 113; h_s = 30 feet

Find: The net force and moment on the sheet-pile wall.

Solution: (1) Determine H_i/gT^2 and H_i/d_s:

$$\frac{H_i}{gT^2} = \frac{10}{(32.2)(6)^2} = 0.00863$$

$$\frac{H_i}{d_s} = \frac{10}{20} = 0.5$$

(2) Find h_o: From Figure 85 for H_i/gT^2 = 0.00863 and H_i/d_s = 0.5:

$$\frac{h_o}{H_i} = 0.49$$

$$h_o = 0.49\ H_i$$

$$h_o = (0.49)\ (10) = 4.9 \text{ feet}$$

(3) Using Equations (5-2) and (5-3), respectively, find S_c and S_t, then determine if wall will be overtopped:

$$S_c = d_s + h_o + H_i = 20 + 4.9 + 10$$

$$S_c = 34.9 \text{ feet}$$

$$S_t = d_s + h_o - H_i = 20 + 4.9 - 10$$

$$S_t = 14.9 \text{ feet}$$

Check to see if $S_c > h_s$:

S_c = 34.9 feet and h_s = 30 feet

$S_c > h_s$; therefore, the structure will be overtopped.

(4) Use Figures 109 and 110 to determine the force and moment, respectively, acting on the seaward side of the wall, in a leeward direction ("leeward-acting") when the incident wave crest is at the seaward side of the structure (see Figure 113A):

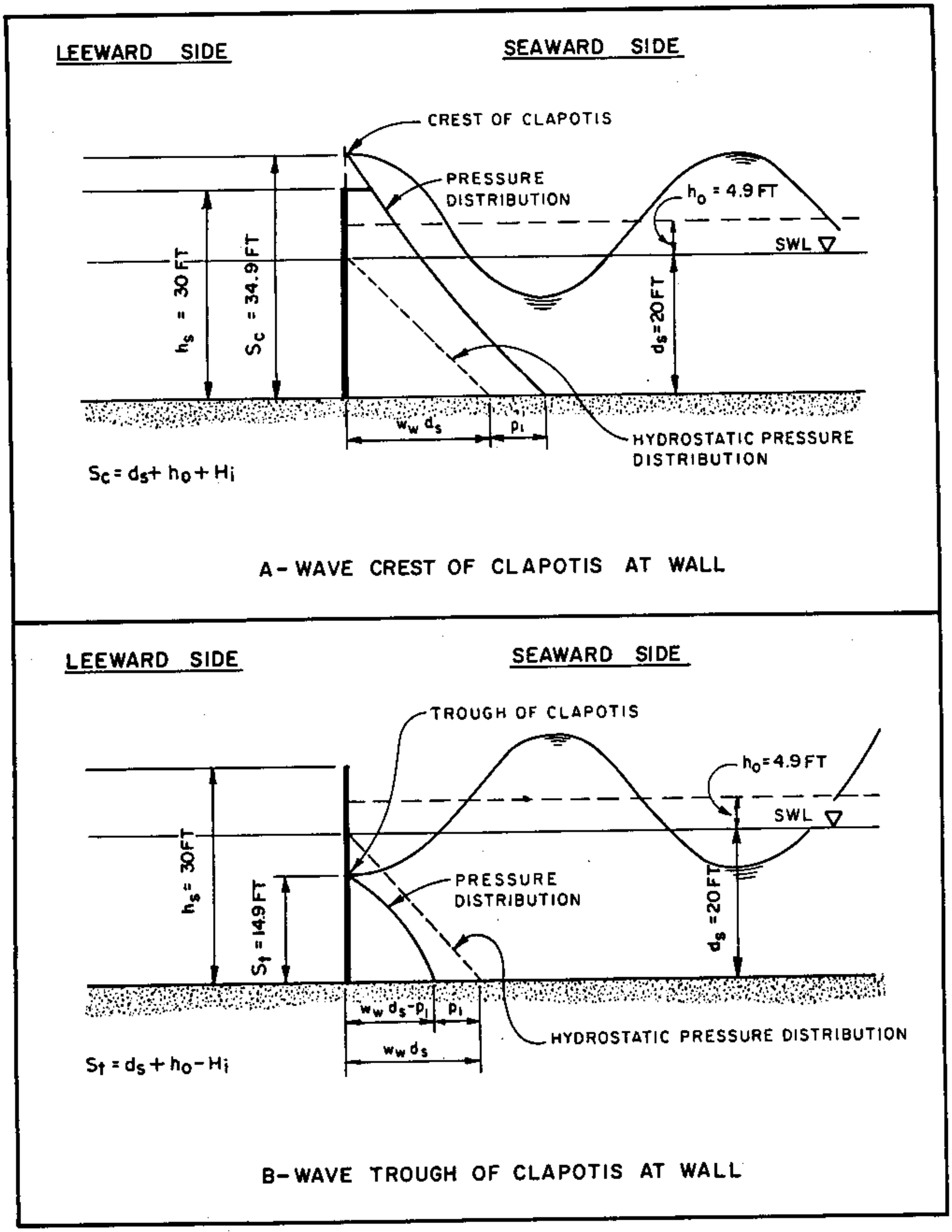


FIGURE 113
Diagrams for Example Problem 30

EXAMPLE PROBLEM 30 (Continued)

(a) Find wave force when incident wave crest is at structure:

From Figure 109 for $H_i/gT^2 = 0.00863$ and $H_i/d_s = 0.5$ (wave crest at structure):

$$\frac{F_c}{w\, d_s^2} = 1.00$$

(Assuming salt water, $w = 64$ pounds per cubic foot.)

$$F_c = (1.00)\,[(64)\,(20)^2]$$

$F_c = 25{,}600$ pounds per foot

Now apply the correction factor for a wall of low height using Equation (5-19); $S = S_c$:

$$r_f = \left(\frac{h_s}{S_c}\right)\left(2 - \frac{h_s}{S_c}\right)$$

$$r_f = \left(\frac{30}{34.9}\right)\left(2 - \frac{30}{34.9}\right)$$

$$r_f = 0.980$$

Using Equation (5-17), find F'_c; $F' = F'_c$ and $F = F_c$:

$$F'_c = r_f\, F_c$$

$$F'_c = (0.980)\,(25{,}600)$$

$F'_c = 25{,}088$ pounds per foot (leeward-acting force with incident crest at seaward side of structure)

(b) Find moment when incident wave crest is at structure:

From Figure 110 for $H_i/gT^2 = 0.00863$ and $H_i/d_s = 0.5$ (wave crest at structure):

$$\frac{M_c}{w\, d_s^3} = 0.55$$

$$M_c = (0.55)\,[\,(64)\,(20)^3]$$

$M_c = 281{,}600$ foot-pounds per foot

EXAMPLE PROBLEM 30 (Continued)

Now apply the correction factor for a wall of low height using Equation (5-20); $S = S_c$:

$$r_m = \left(\frac{h_s}{S_c} \right)^2 \left[3 - 2 \left(\frac{h_s}{S_c} \right)\right]$$

$$r_m = \left(\frac{30}{34.9} \right)^2 \left[3 - 2 \left(\frac{30}{34.9} \right)\right]$$

$$r_m = 0.946$$

Using Equation (5-18), find M'_c; $M' = M'_c$ and $M = M_c$:

$$M'_c = r_m M_c$$

$$M'_c = (0.946)(281{,}600)$$

M'_c = 266,394 foot-pounds per foot (leeward-acting moment with incident crest at structure)

(5) Determine the net force and moment when the incident wave crest is at the seaward side of the structure:

To determine the net force and moment when the crest is at the structure, the seaward-acting force and moment must be determined. The seaward-acting force and moment depend on the wave action present on the lee side of the structure. For the purposes of this example, it is assumed that the only wave action on the lee side of the wall is the transmitted wave. The methods described in Subsection 3.5.a.(1), Vertical-Wall, Vertical-Thin Wall, or Composite Breakwaters, are used to calculate H_t. For conservative design it is assumed that when a wave crest is at a structure on the seaward side, a wave trough occurs simultaneously on the leeward side, and vice versa. Furthermore, in order to ensure conservative design, it is assumed that the transmitted wave has the same wave period as the incident wave. (In nature, the wave period of the transmitted wave would probably be lower than that of the incident wave (Seelig, 1980).)

(a) Find H_t:

$$\frac{h_s - d_s}{H_i} = \frac{30 - 20}{10} = 1.0$$

From Figure 90 for $(h_s - d_s)/H_i = 1.0$:

$$K_t = \frac{H_t}{H_i} = 0.10$$

EXAMPLE PROBLEM 30 (Continued)

$H_t = K_t H_i = (0.10)(10) = 1.0$ foot

(b) Use Figures 109 and 110 to determine the force and moment, respectively, acting on the leeward side of the wall, in a seaward direction ("seaward-acting") when there is a transmitted trough at the leeward side of the wall:

Determine H_t/gT^2 and H_t/d_s:

$$\frac{H_t}{gT^2} = \frac{1.0}{(32.2)(6)^2} = 0.000863$$

$$\frac{H_t}{d_s} = \frac{1.0}{20} = 0.05$$

For consistency of symbols, H_i replaces H_t in the following calculation of transmitted-wave forces and moments.

From Figure 109 for $H_i/gT^2 = 0.000863$ and $H_i/d_s = 0.05$ (wave trough at structure)

$$\frac{F_t}{w_w d_s^2} = 0.46$$

$F_t = (0.46)[(64)(20)^2]$

$F_t = 11,776$ pounds per foot (seaward-acting force with transmitted trough at leeward side of structure):

From Figure 110 for $H_i/gT^2 = H_i/gT^2 = 0.000863$ and $H_i/d_s = H_t/d_s = 0.05$ (wave trough at structure):

$$\frac{M_t}{w_w d_s^3} = 0.15$$

$M_t = (0.15)[(64)(20)^3]$

$M_t = 76,800$ foot-pounds per foot (seaward-acting moment with transmitted trough at leeward side of structure)

(c) Find net force and moment when incident wave crest is at seaward side of structure and transmitted wave trough is at leeward side:

EXAMPLE PROBLEM 30 (Continued)

$$F_{net} = F'_c - F_t$$

$$F_{net} = 25{,}088 - 11{,}776$$

F_{net} = 13,312 pounds per foot (in a leeward direction: leeward-acting)

$$M_{net} = M'_c - M_t$$

$$M_{net} = 266{,}394 - 76{,}800$$

M_{net} = 189,594 foot-pounds per foot (in a leeward direction: leeward-acting)

(6) Use Figures 109 and 110 to determine the force and moment, respectively, acting on the seaward side of the wall, in a leeward direction ("leeward-acting") when the incident wave trough is at the seaward side of the structure (see Figure 113B):

(a) Find wave force when incident wave trough is at structure:

Note: In this case, no correction for low height is necessary when the wave trough is at the structure since S_t = 14.9 feet is less than h_s = 30 feet: $S_t < h_s$.

From Figure 109 for $H_i/gT^2 = 0.00863$ and $H_i/d_s = 0.5$ (wave trough at structure):

$$\frac{F_t}{w_w d_s^2} = 0.21$$

$$F_t = (0.21)[(64)(20)^2]$$

F_t = 5,376 pounds per foot (leeward-acting force with incident trough at seaward side of structure)

(b) Find moment when incident trough is at structure:

From Figure 110 for $H_i/gT^2 = 0.00863$ and $H_i/d_s = 0.5$ (wave trough at structure):

$$\frac{M_t}{w_w d_s^2} = 0.05$$

$$M_t = (0.05)[(64)(20)^3]$$

EXAMPLE PROBLEM 30 (Continued)

M_t = 25,600 foot-pounds per foot (leeward-acting moment with incident trough at structure)

(7) Determine the net force and moment when the incident wave trough is at the seaward side of the structure:

(a) Use Figures 109 and 110 to determine the force and moment, respectively, acting on the leeward side of the wall, in a seaward direction ("seaward-acting") when there is a transmitted crest at the leeward side of the structure:

For consistency of symbols, H_i replaces H_t in the following calculation of transmitted-wave forces and moments.

From Figure 109 for $H_i/g T^2$ = 0.000863 and H_i/d_s = 0.05 (wave crest at structure):

$$\frac{F_c}{w_w d_s^2} = 0.55$$

F_c = (0.55) [(64) (20)^2]

F_c = 14,080 pounds per foot

From Figure 110 for $H_i/g T^2$ = 0.000863 and H_i/d_s = 0.05 (wave crest at structure):

$$\frac{M_c}{w_w d_s^3} = 0.19$$

M_c = (0.19) [(64) (20)^3]

M_c = 97, 280 foot-pounds per foot

(b) Find net force and moment when incident wave trough is at seaward side of structure and transmitted wave crest is at leeward side:

$F_{net} = F_t - F_c$

F_{net} = 5,376 - 14,080

F_{net} = -8,704 pounds per foot (in a seaward direction: seaward-acting)

$M_{net} = M_t - M_c$

EXAMPLE PROBLEM 30 (Continued)

MÚnet¿ = 25,600 - 97,280

MÚnet¿ = -71,680 foot-pounds per foot (in a seaward direction: seaward-acting)

Therefore, when the wave crest is at the seaward side of the structure, the design force and moment will be at a maximum.

(8) Force, F", and moment, M", on a wall built on a rubble base (Figure 114) or on a baffle with a gap at the bottom can be calculated as follows:

F" = (1 - rÚf¿) F (5-21)

and

M"ÚA¿ = (1 - rÚm¿) M (5-22)

WHERE: F" = force on wall built on rubble base or force on baffle

$$M''_A = (1 - r_m)\ M \qquad (5\text{-}22)$$

WHERE: F" = force on wall built on rubble base or force on baffle

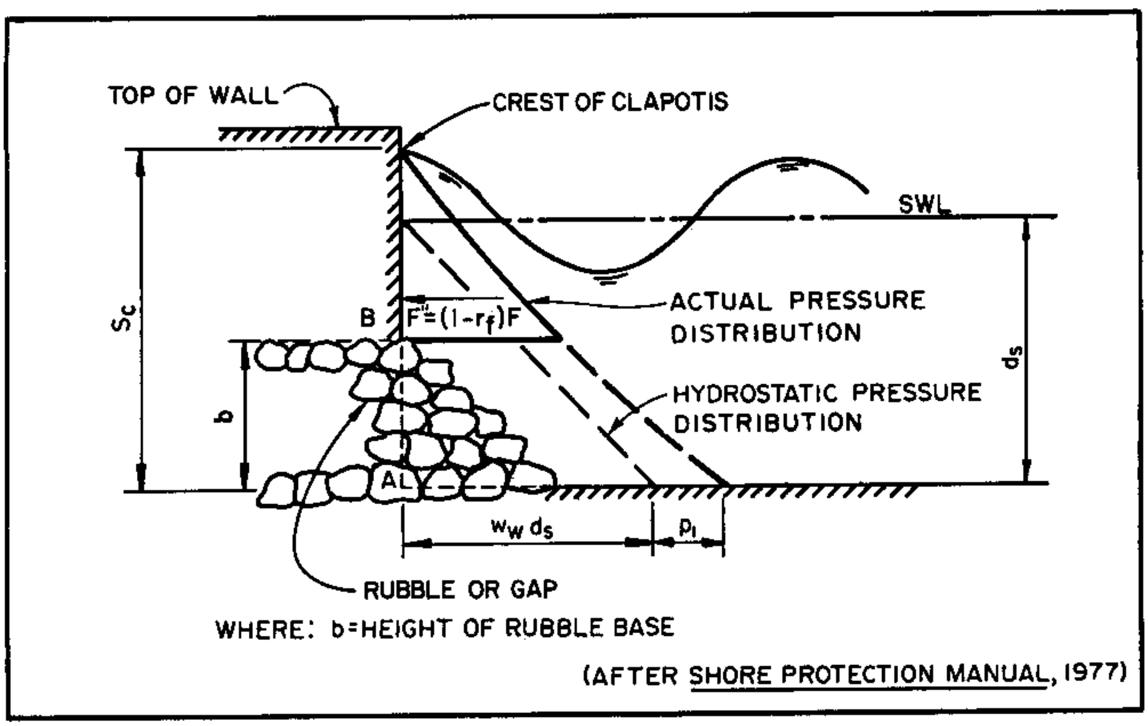


FIGURE 114
Pressure Distribution for Wall Built on Rubble Base

r_f, r_m = reduction factors determined using Equations (5-19) and (5-20) using b (the height above the bottom of the rubble base or of the gap) instead of h_s (the height of the wall) (see Figure 114)

F = force determined for "ordinary" wall

M''_A = moment about the mudline (A in Figure 114) for wall built on rubble base

M = moment about the mudline determined for "ordinary" wall

The moment about the base of the wall (B in Figure 114) is determined by:

$$M''_B = M''_A - b\,F'' \qquad (5\text{-}23)$$

WHERE: M''_B = moment about the base of the wall (B in Figure 114)

b = height (above the bottom) of rubble base or of gap

EXAMPLE PROBLEM 31

Given:
a. Incident wave height, H_i = 5 feet
b. Water depth, d_s = 35 feet
c. Wave period, T = 4 seconds
d. Wave-baffle structure as shown in Figure 115; distance from SWL to bottom of structure, h = 17 feet, height of gap, b = 18 feet, and height of structure, h_s = 42 feet

Find: The force and moment on the structure, when the wave crest is at the structure.

Solution: (1) Determine H_i/gT^2 and H_i/d_s:

$$\frac{H_i}{g\,T^2} = \frac{5}{(32.2)\,(4)^2} = 0.00970$$

$$\frac{H_i}{d_s} = \frac{5}{35} = 0.14$$

(2) Find h_o:

From Figure 85 for H_i/gT^2 = 0.00970 and H_i/d_s = 0.14:

$$\frac{h_o}{H_i} = 0.2$$

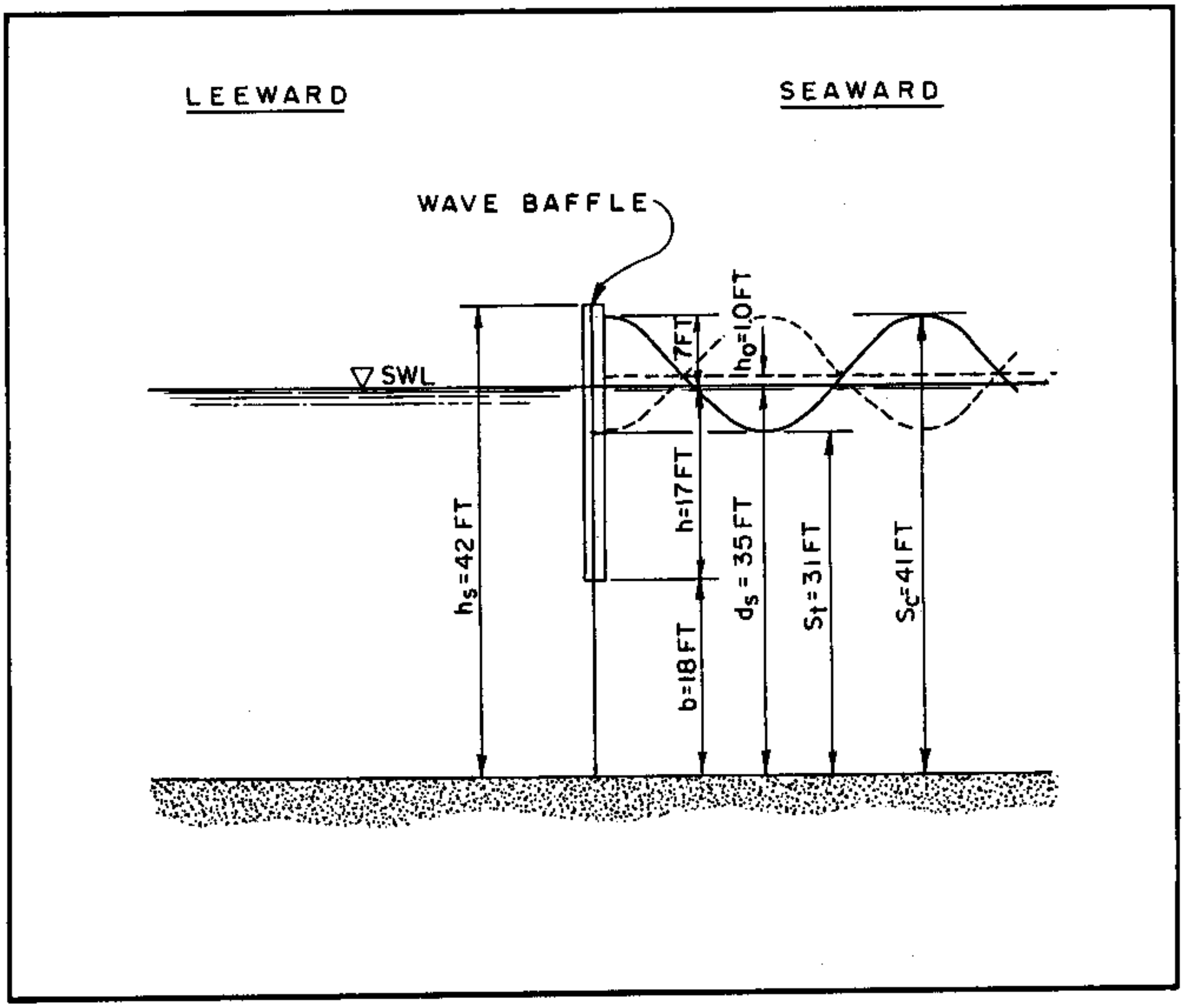


FIGURE 115
Diagram for Example Problem 31

EXAMPLE PROBLEM 31 (Continued)

$h_o = (0.2)\ (5) = 1$ foot

(3) Using Equations (5-2) and (5-3), respectively, find S_c and S_t; then determine if baffle will be overtopped:

$$S_c = d_s + h_o + H_i = 35 + 1 + 5$$

$$S_c = 41 \text{ feet}$$

$$S_t = d_s + h_o - H_i = 35 + 1 - 5$$

$$S_t = 31 \text{ feet}$$

Check to see if $S_c > h_s$:

$S_c = 41$ feet and $h_s = 42$ feet

$S_c < h_s$; therefore, the structure will not be overtopped.

(4) Find force and moment when wave crest is at baffle (wave crest at structure):

(a) From Figure 109 for $H_i/g T^2 = 0.00970$ and $H_i/d_s = 0.14$:

$$\frac{F_c}{w_w\ d_s^3} = 0.53$$

(Assuming salt water, $w_w = 64$ pounds per cubic foot.)

$$F_c = (0.53)\ [\ (64)\ (35)^2]$$

$$F_c = 41{,}552 \text{ pounds per foot}$$

(b) Correct for the fact that the structure does not extend to the bottom:

Using Equation (5-19), find r_f; $S = S_c$ and $h_s = b$:

$$r_f = \left(\frac{b}{S_c} \right) \left(2 - \frac{b}{S_c} \right)$$

$$r_f = \left(\frac{18}{41} \right) \left(2 - \frac{18}{41} \right)$$

$$r_f = 0.69$$

Using Equation (5-21), find F''_c; $F'' = F''_c$ and $F = F_c$:

$$F''_c = (1 - r_f)\ F_c$$

EXAMPLE PROBLEM 31 (Continued)

$$F''_c = (1 - 0.69)(41{,}552) = 12{,}881 \text{ pounds per foot}$$

(c) Find F_c when the trough of the transmitted wave acts on the leeward side of wall:

Determine the force acting on the leeward side (seaward-acting). This is done by determining the wave transmission through the baffle using Figure 92 in Section 3, BASIC PLANNING. In order to use this figure, first determine d/L and h/d_s:

First, find d/L:

$$L_o = (g/2\pi)\, T^2 = (32.2/2\pi)(4)^2 = 82.0 \text{ feet}$$

$$\frac{d_s}{L_o} = \frac{35}{82.0} = 0.427$$

From Figure 2 for $d_s/L_o = 0.427$:

$$\frac{d}{L} = 0.43$$

Now, find h/d_s, where h = distance from water surface (SWL) to bottom of structure; refer to Figure 115 for the given value of h:

$$\frac{h}{d_s} = \frac{17}{35} = 0.485$$

From Figure 92 for $h/d_s = 0.485$ and $d_s/L = 0.437$:

$$K_t = \frac{H_t}{H_i} = 0.3$$

$$H_t = K_t H_i = (0.3)(5) = 1.5 \text{ feet}$$

Use Figure 109 to determine the force acting on the leeward side of the wall. Determine $H_t/g T^2$ and H_t/d_s:

$$\frac{H_t}{g T^2} = \frac{1.5}{(32.2)(4)^2} = 0.00291$$

$$\frac{H_t}{d_s} = \frac{1.5}{35} = 0.0429$$

For consistency of symbols, H_i replaces H_t in the

EXAMPLE PROBLEM 31 (Continued)

following calculation of transmitted-wave forces and moments.

From Figure 109 for $H_i/gT^2 = 0.00291$ and $H_i/d_s = 0.0429$ (wave trough at structure):

$$\frac{F_t}{w_w d_s^2} = 0.48$$

(Assuming salt water, w_w = 64 pounds per cubic foot.)

$$F_t = (0.48)\ [\ (64)\ (35)^2]$$

F_t = 37,632 pounds per foot

From Figure 85 for $H_i/gT^2 = 0.00291$ and $H_i/d_s = 0.0429$:

$$\frac{h_o}{H_i} = 0.03$$

$h_o = (0.03)\ (1.5) = 0.045$ feet

Using Equation (5-3) with $H_i = H_t$, find S_t:

$$S_t = d_s + h_o - H_t = 35 + 0.045 - 1.5$$

S_t = 33.55 feet

Use Equation (5-19) for r_f with $S = S_t$ in order to correct for the fact that the structure does not extend to the bottom; $h_s = b$:

$$r_f = \left(\frac{b}{S_t}\right)\left(2 - \frac{b}{S_t}\right)$$

$$r_f = \left(\frac{18}{33.55}\right)\left(2 - \frac{18}{33.55}\right)$$

$$f_f = 0.79$$

Using Equation (5-21), find F''_t; $F'' = F''_t$ and $F = F_t$:

$$F''_t = (1 - r_f)\ F_t$$

$F''_t = (1 - 0.79)\ (37,632) = 7,903$ pounds per foot

(d) Find F_{net} when the wave crest is at the seaward side of the wall:

EXAMPLE PROBLEM 31 (Continued)

$$F''_{net} = F''_c - F''_t$$

$$F''_{net} = 12,881 - 7,903$$

$F''_{net} = 4,978$ pounds per foot

(e) From Figure 110 for $H_i/gT^2 = 0.00970$ and $H_i/d_s = 0.14$ (wave crest at structure):

$$\frac{M_c}{w_w\, d_s^3} = 02.0$$

$$M_c = (0.20)\ [(64)\ (35)^3]$$

$M_c = 548,800$ foot-pounds per foot

(f) Correct for the fact that the structure does not extend to the bottom:

Using Equation (5-20), find r_m; $S = S_c$ and $h_s = b$:

$$r_m = \left(\frac{b}{S_c} \right) \left[3 - 2 \left(\frac{b}{S_c} \right) \right]$$

$$r_m = \left(\frac{18}{41} \right)^2 \left[3 - 2 \left(\frac{18}{41} \right) \right]$$

$r_m = 0.41)$

Using Equation (5-22), find M''_c; $M''_A = M''_c$ and $M = M_c$:

$$M''_c = (1 - r_m)M_c$$

$$M''_c = (1 - 0.41)\ (548,800)$$

$M''_c = 323,792$ foot-pounds per foot

(g) Find net moment:

Use Figure 110 for $H_i/gT^2 = H_t/gT^2 = 0.00291$ and $H_i/d_s = H_t/d_s = 0.0429$ to determine the moment acting on the leeward side of the wall (wave trough at structure):

$$\frac{M_t}{w_w\, d_s^3} = 0.158$$

$$M_t = (0.158)\ [\ (64)\ (35)^3]$$

MÚt¿ = 433,552 foot-pounds per foot

EXAMPLE PROBLEM 31 (Continued)

Using Equation (5-20), find r_m; $S = S_t$ and $h_s = b$:

$$r_m = \left(\frac{b}{S_t} \right)^2 \left[3 - 2 \left(\frac{b}{S_t} \right) \right]$$

$$r_m = \left(\frac{18}{33.55} \right)^2 \left[3 - 2 \left(\frac{18}{33.55} \right) \right]$$

$$r_m = 0.55$$

Using Equation (5-22), find M''_t; $M''_A = M''_t$ and $M = M_t$:

$$M''_t = (1 - r_m)\, M_t$$

$$M''_t = (1 - 0.55)\,(433{,}552)$$

M''_t = 195,098 foot-pounds per foot

(h) Find net moment M''_{net}:

$$M''_{net} = M''_c - M''_t$$

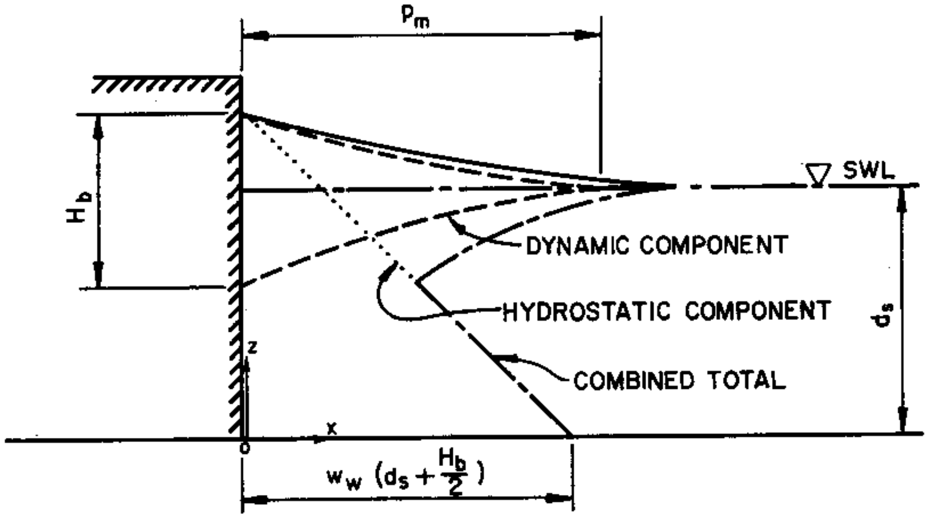
$$M''_{net} = 323{,}792 - 195{,}098$$

M''_{net} = 128,694 foot-pounds per foot

(9) In cases for nonvertical walls and for waves at angles of attack other than normal to the wall, forces are assumed to be similar to those for the case of normal wave attack.

c. Breaking Waves. Breaking waves exert hydrostatic forces and dynamic-impact forces on vertical walls. The hydrostatic forces should he used in design for preventing sliding and overturning. Dynamic-impact forces occur at the instant when the vertical front face of a breaking wave impinges on the wall, and only when a plunging wave traps a cushion of air against the wall. The dynamic-impact force occurs only on smooth-faced structures. The dynamic-impact force can be an order-of-magnitude greater than the hydrostatic force; however, it is applied on the order of 1/100 second over a small area. Because walls have a shock-absorbing capacity, they need not be designed for sliding and overturning using dynamic-impact forces. However, these forces should be considered when the wall is made of small or weak units, such as blocks. In such a case, impact forces may cause some damage to the structural components. Therefore, when designing in the region of breaking waves, the designer should calculate the hydrostatic force and determine if the dynamic-impact forces are applicable to the type of construction proposed.

When breaking waves impinge on a wall, reflection is assumed to be negligible and the pressures are both hydrostatic and dynamic. Figure 116 gives the pressure distribution for hydrostatic and dynamic-impact forces.



NOTE: EQUATIONS PRESENTED IN SECTION 1 ARE BASED ON THE z-COORDINATE AXIS WITH ITS ORIGIN AT THE STILL WATER LEVEL. EQUATIONS IN SECTIONS 5 AND 7 ARE BASED ON THE z-COORDINATE AXIS WITH ITS ORIGIN AT THE BOTTOM.

(AFTER SHORE PROTECTION MANUAL, 1977)

FIGURE 116
Pressure Distribution for Breaking Waves at a Wall
(Minikin Wave-Pressure Diagram)

(1) Hydrostatic Force and Moment.

(a) Pressure. The hydrostatic pressure is given by:

$$p = 0 \qquad \text{at } z = S_c = d_s + \frac{H_b}{2} \qquad (5\text{-}24)$$

and

$$p = w_w \left(d_s + \frac{H_b}{2}\right) \qquad \text{at } z = 0 \qquad (5\text{-}25)$$

WHERE:
- p = pressure
- z = vertical distance along a coordinate axis with its origin at the bottom
- S_c = depth from wave crest
- H_b = breaking-wave height
- d_s = depth at structure toe from SWL
- w_w = unit weight of water

The hydrostatic pressure distribution is assumed to be linear between S_c and $z = 0$ (see Figure 116).

(b) Force and moment. The total hydrostatic force, F_s, on a nonovertopped wall is:

$$F_s = \frac{1}{2} \; w_w \left(d_s + \frac{H_b}{2}\right) \qquad (5\text{-}26)$$

WHERE: F_s = hydrostatic component of force for breaking wave and the hydrostatic moment, M_s, is:

$$F_s = \frac{1}{6} \; w_w \left(d_s + \frac{H_b}{2}\right)^3 \qquad (5\text{-}27)$$

WHERE: M_s = hydrostatic component of moment for breaking wave

(2) Dynamic Force and Moment. Equations for dynamic-impact, or shock, force, F_m, and moment, M_m, due to a breaking wave acting on a smooth vertical wall are given below.

(a) Pressure. The maximum dynamic pressure, assumed to act at the still water level, is given by the Minikin equation:

$$p_m \; 101 \; w_w \left(\frac{H_b}{L_D}\right)\left(\frac{d_s}{D}\right)(D + d_s) \qquad (5\text{-}28)$$

WHERE: p_m = maximum dynamic pressure

w_w = unit weight of water

H_b = breaking-wave height

L_D = wavelength at depth D

d_s = water depth at structure toe from SWL

D = water depth one wavelength seaward of the wall

The depth, D, one wavelength seaward of the wall (where the wavelength, L_{d_s} is determined using the depth at the wall, d_s) may be determined by the following equation:

$$D = d_s + L_{d_s} m \qquad (5\text{-}29)$$

WHERE: L_{d_s} = wavelength in a depth equal to d_s
m = nearshore bottom slope

The dynamic pressure distribution is shown in Figure 116. The dynamic pressure is assumed to act between $z = d_s + (H_b/2)$ and $z = d_s - (H_b/2)$.

(b) Force and moment. The integrated dynamic-impact force is:

$$F_m = \frac{p_m H_b}{3} \qquad (5\text{-}30)$$

WHERE: F_m = dynamic component of force for breaking wave
and the moment about the mudline is:

$$M_m = F_m d_s \qquad (5\text{-}31)$$

WHERE: M_m = dynamic component of moment for breaking wave

(3) Total Force and Moment. The total force, F_T, is obtained by adding the hydrostatic force, F_s, to the dynamic force, F_m:

$$F_T = F_s + F_m \qquad (5\text{-}32)$$

The total moment, M_T, is obtained by adding the hydrostatic moment, M_s, to the dynamic moment, M_m:

$$M_T = M_s + M_m \qquad (5\text{-}33)$$

(4) Wall Fronted by a Sloping Bottom. Equations (5-30) and (5-31) are solved in Figure 117 as a function of $d_s/g T^2$ and slope, m. F_m and M_m are determined by first calculating $d_s/g T^2$ (or d_s/T^2) and finding the parameters $p_m/(w_w H_b)$ and $3 F_m/(w_w H_b)$; then

$$p_m = (\text{value from Figure 117})\ (w_w H_b) \qquad (5\text{-}34)$$

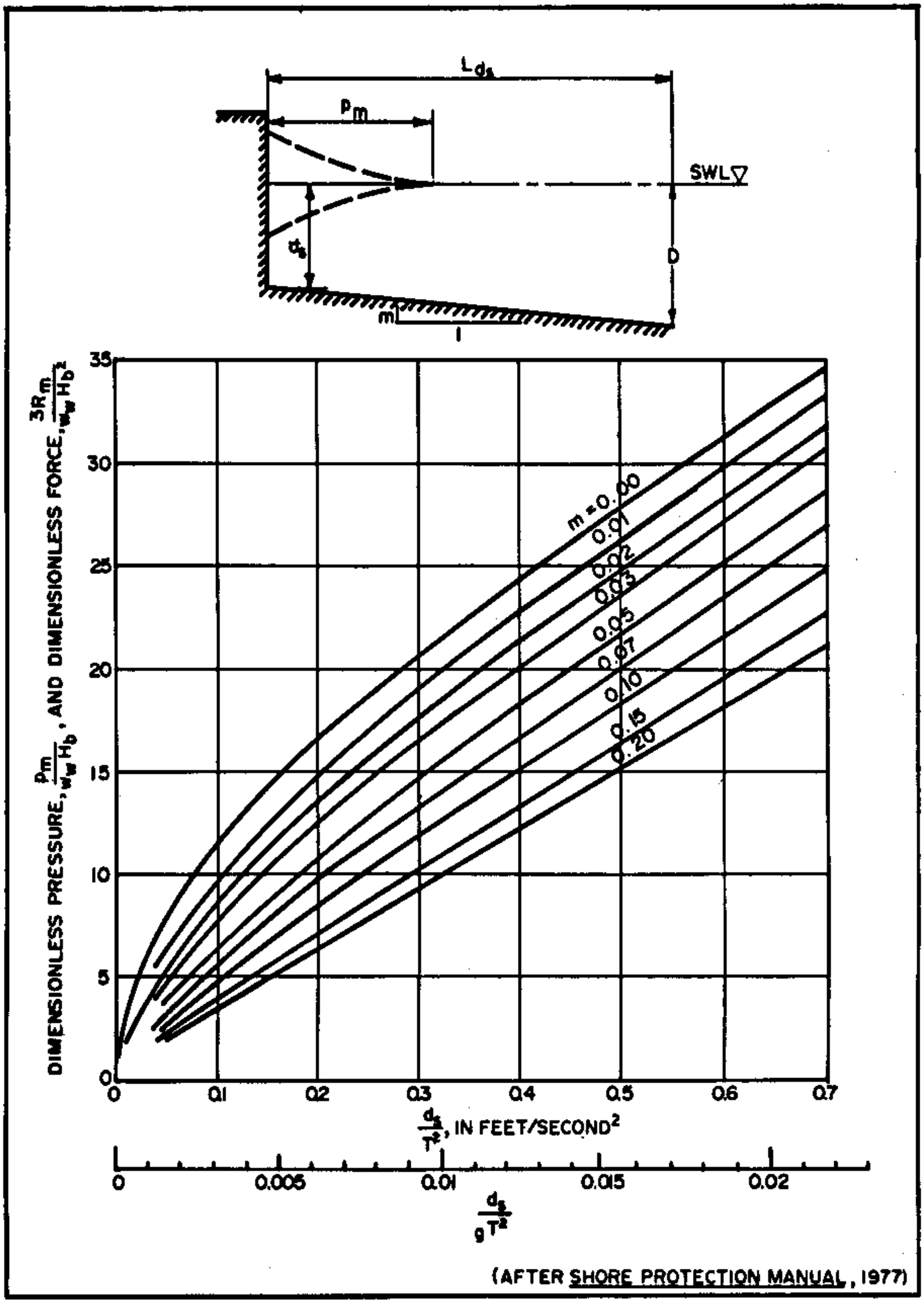


FIGURE 117
Dimensionless Wave Pressure and Force for a Wall Fronted by a Sloping Bottom

$$F_m = [(\text{value from Figure 117})(w_w H_b^2)]/3 \qquad (5\text{-}35)$$

$$M_m = F_m d_s \qquad (5\text{-}36)$$

Total forces and moments are then determined using Equations (5-32) and (5-33). (Hydrostatic forces are determined as explained previously.)

(5) Wall on a Rubble Foundation. Figure 118 solves Equations (5-30) and (5-31) for a wall built on a rubble foundation as a function of d_s $/g T^2$ (or d_s/T^2) and the ratio of the water depth at the wall, d_w, to water depth at the toe of the rubble foundation, d_s: d_w/d_s. Solve for p_m, F_m, and M_m by Equations (5-34), (5-35), and (5-36).

Reduced hydrostatic wave forces and moments for walls on a rubble foundation can be calculated using Equations (5-21) and (5-22), respectively. Values for r_f and r_m are obtained using Equations (5-19) and (5-20), respectively, where S equals S_c (which is $d_s + (H_b/2)$) or S_t (which is d_s $(H_b/2)$) and $h_s = b$ = height of rubble base.

Total forces and moments are then determined using Equations (5-32) and (5-33).

(6) Wall of Low Height. For a low-height wall (when the top of the wall is lower than the height of the design breaking-wave crest), the force and moment are corrected by using a force-reduction factor, r_m. Figure 119 gives values for r_m for different values of b'/H_b. (b' is defined in Figure 119.) Then:

$$F'_m = r_m F_m \qquad (5\text{-}37)$$

WHERE: F'_m = corrected dynamic-impact force for overtopped wall

r_m = reduction factor for dynamic-impact force (determined from Figure 119)

For the moment, M'_m, an additional reduction factor, a (from Figure 120), is required for use in the equation:

$$M'_m = d_s F_m - (d_s + a)(1 - r_m) F_m$$

which reduces to:

$$M'_m = F_m [r_m (d_s + a) - a] \qquad (5\text{-}38)$$

WHERE: M'_m = corrected dynamic moment about the mudline for overtopping breaking wave

a = reduction factor for dynamic moment (determined from Figure 120)

Reduced hydrostatic wave forces and moments, F'_s and M'_s, respectively, for a wall of low height can be calculated using Equations 5-17) and (5-18). Values for r_f and r_m are obtained using Equations (5-19) and (5-20), respectively, with $S = S_c = d_s + H_b/2$.

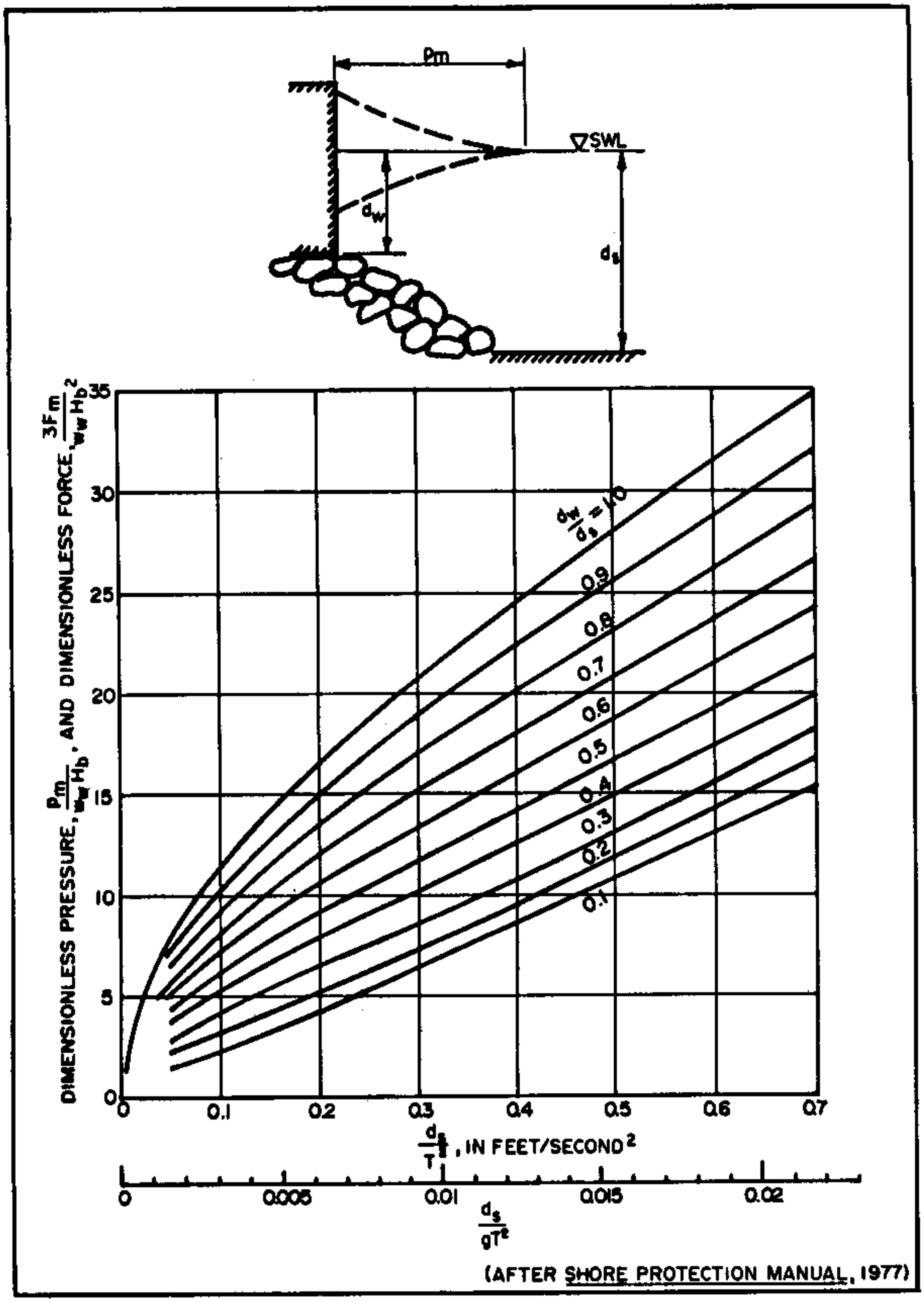


FIGURE 118

Dimensionless Wave Pressure and Force for a Wall on a Rubble Foundation

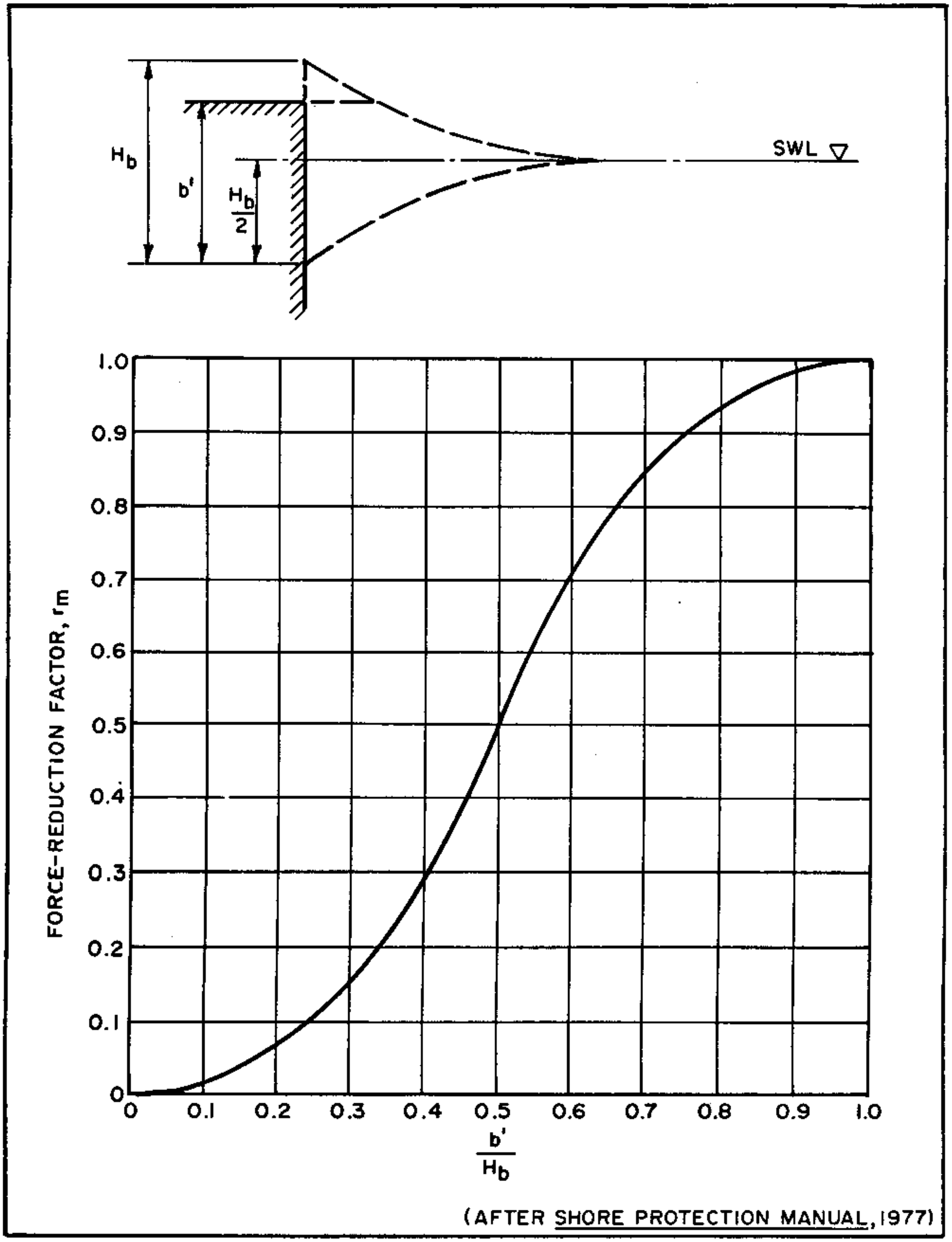


FIGURE 119
Force-Reduction Factor for Low-Height Wall

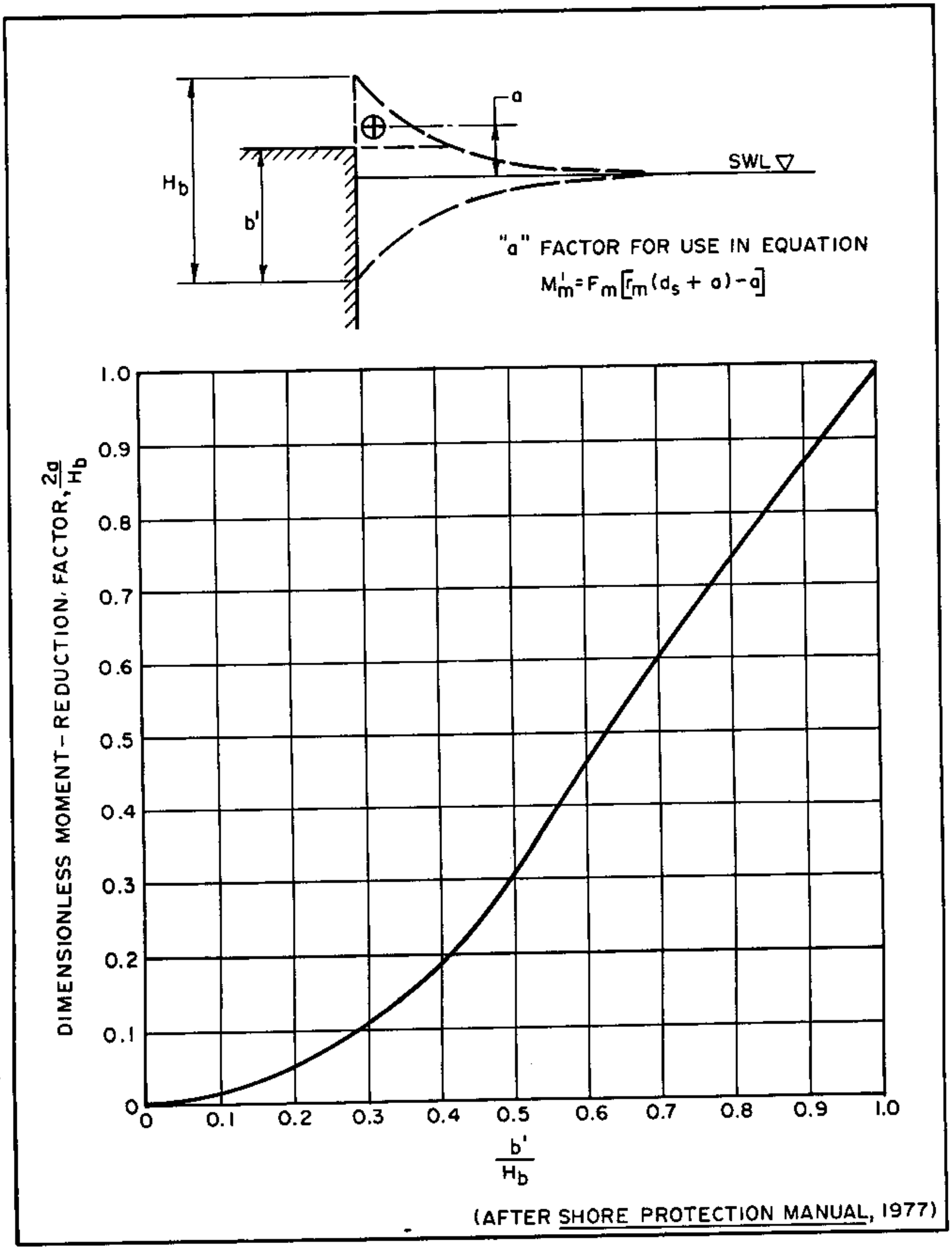


FIGURE 120
Moment-Reduction Factor for Low-Height Wall

EXAMPLE PROBLEM 32

Given:
a. Breaking-wave height, H_b = 6.6 feet
b. Water depth at structure toe, d_s = 6.0 feet
c. Wave period, T = 5 seconds
d. Slope in front of wall, m = 0.05
e. Vertical wall as shown in Figure 121; h_s = 8 feet and b' = 5.3 feet

Find: Breaking-wave force on the vertical wall.

Solution: (1) Determine hydrostatic Force and moment:

Using Equation (5-26), find F_s:

$$F_s = \frac{1}{2} w_w \left(d_s + \frac{H_b}{2} \right)^2$$

(Assuming salt water, w_w = 64 pounds per cubic foot.)

$$F_s = \left(\frac{1}{2}\right) (64) \left(6.0 + \frac{6.6}{2}\right)^2$$

F_s = 2,768 pounds per foot

Using Equation (5-19), find r_f; $S_c = d_s + H_b/2$:

$$r_f = \left(\frac{h_s}{S_c} \right) \left(2 - \frac{h_s}{S_c}\right)$$

$$S_c = d_s + \frac{H_b}{2}$$

$$S_c = 6.0 + \frac{6.6}{2} = 9.3$$

$$r_f = \left(\frac{8}{9.3} \right) \left(2 - \frac{8}{9.3}\right)$$

r_f = 0.98

Using Equation (5-17), find F'_s; $F' = F'_s$ and $F = F_s$:

$F'_s = r_f F_s$

F'_s = (0.98) (2,768)

F'_s = 2,713 pounds per foot

Using Equation (5-27):

$$M_s = \frac{1}{6} w_w \left(d_s + \frac{H_b}{2}\right)^3$$

$$r_f = \left(\frac{1}{6}\right)(64)\left(6.0 + \frac{6.6}{2}\right)^3$$

$$M_s = 8{,}580 \text{ foot-pounds per foot}$$

EXAMPLE PROBLEM 32 (Continued)

Using Equation (5-20), find r_m; $S_c = d_s + H_b/2 = 9.3$:

$$r_m = \left(\frac{h_s}{S_c} \right) \left(3 - 2 \frac{h_s}{S_c}\right)$$

$$r_f = \left(\frac{8}{9.3} \right)^2 \left(3 - 2 \frac{8}{9.3}\right)$$

$$r_m = 0.95$$

Using Equation (5-18), find M'_s; $M' = M'_s$ and $M = M_s$:

$$M'_s = r_m M_s$$

$$M'_s = (0.95)(8{,}580)$$

M'_s = 8,151 foot-pounds per foot

(2) Determine dynamic force and moment:

$$\frac{d_s}{T^2} = \frac{6.0}{(5)^2} = 0.24$$

From Figure 117 for $d_s/T^2 = 0.24$ and $m = 0.05$:

EXAMPLE PROBLEM 32 (Continued)

$$\frac{3 F_m}{w_w H_b^2} = 12.5$$

Using Equation (5-35), find F_m:

F_m = [(value from Figure 117) $(w_w H_b^2)$]/3

$$F_m = \frac{(12.5)(64)(6.6)^2}{3} = 11,616 \text{ pounds per foot}$$

As shown in Figure 121, the wall is overtopped; thus, the force-reduction factor, r_m, should be applied to find the force as reduced by overtopping.

From Figure 121, b' = 5.3

$$\frac{b'}{H_b} = \frac{5.3}{6.6} = 0.80$$

From Figure 119 for $b'/H_b = 0.80$:

$r_m = 0.93$

Using Equation (5-37), find F'_m

$F'_m = r_m F_m$

$F'_m = (0.93)(11,616) = 10,803$ pounds per foot

The moment-reduction factor will also have to be applied:

From Figure 120 for $b'/H_b = 0.80$:

$$\frac{2a}{H_b} = 0.73$$

$$a = \frac{(0.73)(6.6)}{2} = 2.4$$

Using Equation (5-38), find M'_m

$M'_m = (F_m)[r_m (d_s + a) - a]$

$M'_m = (11,616)[(0.93)(6.0 + 2.4) - 2.4]$

$M'_m = 62,866$ foot-pounds per foot

(3) Find total force and moment:

EXAMPLE PROBLEM 32 (Continued)

Using Equations (5-32) and (5-33), respectively, find F_T and M_T

$F_T = F_s + F_m$, where $F_m = F'_m$

$F_T = 2{,}714 + 10{,}803$

$F_T = 13{,}517$ pounds per foot

$M_T = M_s + M_m$, where $M_m = M'_m$

$M_T = 8{,}124 + 62{,}866$

$M_T = 70{,}990$ foot-pounds per foot

d. Broken Waves. Calculation of wave pressures for structures subjected to breaking waves is done by making a series of simple assumptions which are not necessarily consistent with empirical wave-transformation data. The structure may be seaward or shoreward of the still water line.

(1) Wall Seaward of Still Water Line. (See Figure 122.)

(a) Pressure. The hydrostatic pressure is:

$$p_s = w_w\,(d_s + h_c) \qquad (5\text{-}39)$$

WHERE: p = maximum hydrostatic pressure

w_w = unit weight of water

d_s = depth at structure toe from SWL

$h_c = 0.78\,H_b$ = height of broken wave above SWL (5-40)

H_b = breaking-wave height (obtained from Figure 42)

and the dynamic pressure is:

$$p_m = \frac{w_w d_b}{2} \qquad (5\text{-}41)$$

WHERE: p_m = maximum dynamic pressure

w_w = unit weight of water

d_b = depth of water at breaking

(b) Broken-wave forces. The hydrostatic broken-wave force is:

$$F_s = \frac{w_w\,(d_s + h_c)^2}{2} \qquad (5\text{-}42)$$

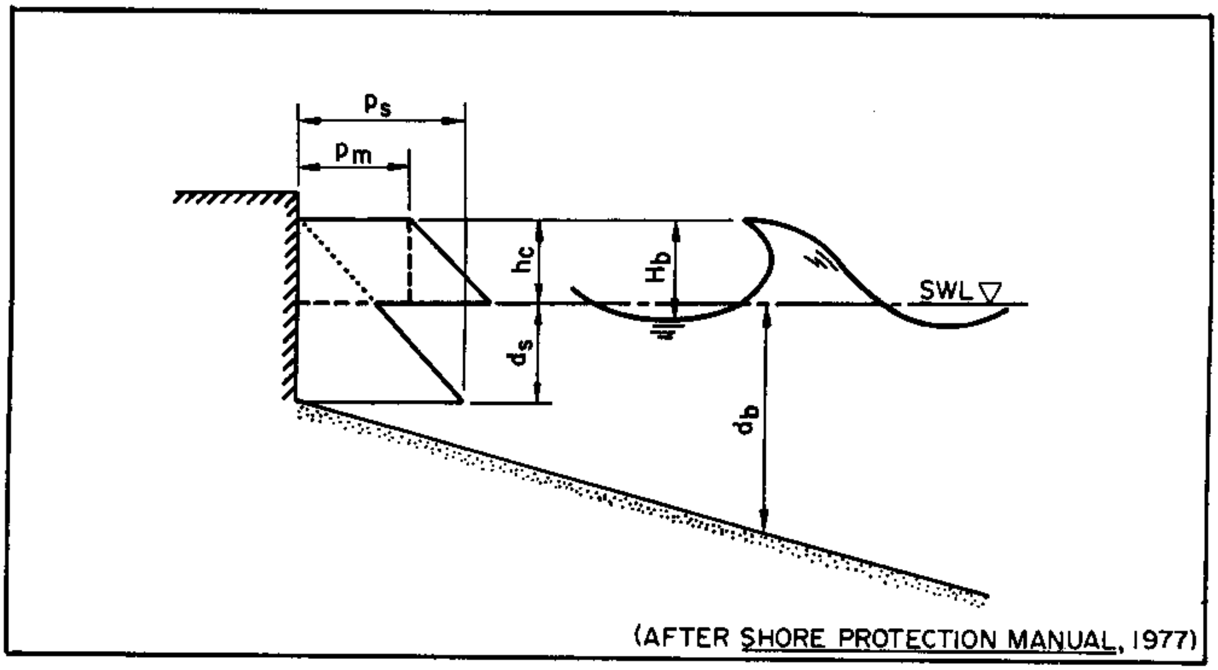


FIGURE 122
Wave Pressures From Broken Waves--Wall Seaward of Still Water Line

Water Line]

WHERE: F_s = hydrostatic component of force for broken wave and the dynamic breaking-wave force is:

$$F_m = p_m h_c \tag{5-43}$$

WHERE: F_m = dynamic component of force for broken wave

(c) Broken-wave moment. The hydrostatic moment is:

$$M_s = \frac{w_w (d_s + h_c)^3}{6} \tag{5-44}$$

WHERE: M_s = hydrostatic component of moment for broken wave and the dynamic moment is:

$$M_m = F_m \left(d_s + \frac{h_c}{2}\right) \tag{5-45}$$

WHERE: M_m = dynamic component of moment for broken wave

(d) Total forces and moments. Total forces and moments are given by:

$$F_T = F_s + F_m \tag{5-46}$$

$$M_T = M_s + M_m \qquad (5\text{-}47)$$

(2) Wall Shoreward of Still Water Line. (See Figure 123.) Structures shoreward of the still water line are subject to hydrostatic and dynamic forces due to wave runup.

(a) Wave runup. Calculate wave runup by methods outlined in Section 3.4., WAVE RUNUP, or conservatively assume:

$$R = 2\ H_b \qquad (5\text{-}48)$$

WHERE: R = runup

H_b = breaking-wave height

(b) Wave height. Find the height of the wave on the wall, h', from:

$$h' = h_c\left(1 - \frac{x_1}{x_2}\right) \qquad (5\text{-}49)$$

WHERE: x_1 = distance from the still water line to the structure

$x_2 = 2\ H_b \cot [\text{beta}] = 2\ H_b/m$ = distance from the still water line to the limit of wave uprush

[beta] = angle of beach slope (see Figure 123)

m = tan [beta]

(c) Pressure. The maximum hydrostatic pressure is:

$$p_s = w_w\ h' \qquad (5\text{-}50)$$

The maximum dynamic pressure, assumed to act uniformly over the height, h', is:

$$p_m = \left(\frac{w_w\ d_b}{2} \right) \left(1 - \frac{x_1}{x_2} \right)^2 \qquad (5\text{-}51)$$

(d) Forces. The total hydrostatic Force is:

$$F_s = \left(\frac{w_w\ h_b^2}{2} \right) \left(1 - \frac{x_1}{x_2} \right)^2 \qquad (5\text{-}52)$$

WHERE: F_s = hydrostatic component of force for broken wave and the dynamic force is:

$$\left(w_w\ d_b\ h_c \right) \left(\quad x_1 \quad \right)^3$$

```
FÚm¿ = ³ ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ  ³ ³ 1 - ÄÄÄÄÄ ³                 (5-53)
       ³        2         ³ ³     xÚ2¿  ³
       ÀÄ                ÄÙ ÀÄ         ÄÙ
```

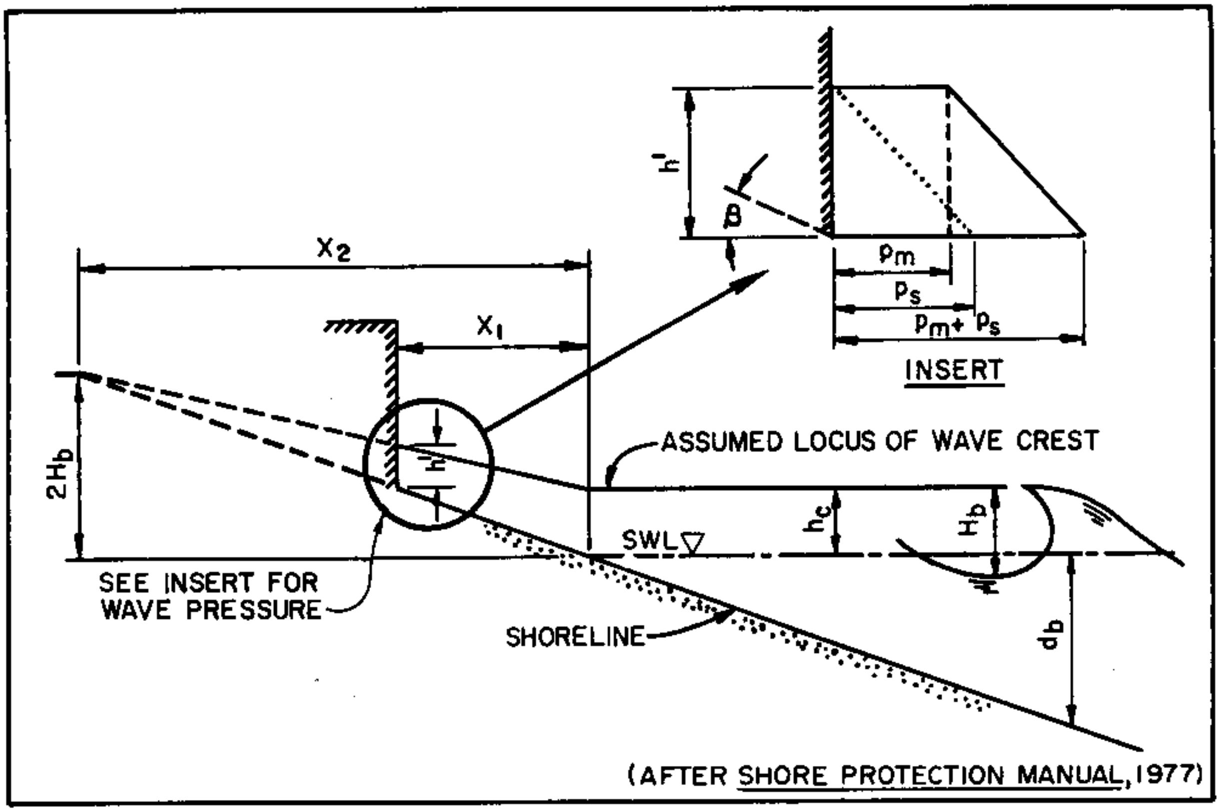


FIGURE 123
Wave Pressures From Broken Waves--Wall Landward of Still Water Line

WHERE: F_m = dynamic component of force for broken wave

The total force is:

$$F_T = F_s + F_m \tag{5-54}$$

(e) Resulting moments. The hydrostatic moment is:

$$M_s = \left[\frac{w_w h_c^{3}}{6} \right] \left[1 - \frac{x_1}{x_2} \right]^{3} \tag{5-55}$$

WHERE: M_s = hydrostatic component of moment for broken wave and the dynamic moment is:

$$M_m = \left[\frac{w_w d_b^{1} h_c^{2}}{4} \right] \left[1 - \frac{x_1}{x_2} \right]^{4} \tag{5-56}$$

WHERE: M_m = dynamic component of moment for broken wave

The resultant moment is:

$$M_T = M_s + M_m \qquad (5\text{-}57)$$

EXAMPLE PROBLEM 33

Given:
a. Breaking-wave height, $H_b = 7.0$ feet
b. Depth at breaking, $d_b = 6.4$ feet
c. Water depth at structure toe, d = 5.0 feet
d. Vertical, smooth-faced wall situated seaward of the still water line.

Find: The broken-wave force on the wall for a normally incident wave.

Solution: (1) Using Equation (5-40), find h_c:

$$h_c = 0.78\, H_b$$

$$h_c = (0.78)(7.0) = 5.5 \text{ feet}$$

(2) Find hydrostatic force:

Using Equation (5-42), find F_s:

$$F_s = \frac{w_w\,(d_s + h_c)^2}{2}$$

(Assuming salt water, w_w = 64 pounds per cubic foot.)

$$F_s = \frac{(64)(5.0 + 5.5)^2}{2}$$

F_s = 3,528 pounds per foot

(3) Find dynamic force:

Using Equations (5-41) and (5-43), respectively, find p_m and F_m:

$$p_m = \frac{w_w\, d_b}{2}$$

$$p_m = \frac{(64)(6.4)}{2}$$

p_m = 205 pounds per square foot

$$F_m = p_m\, h_c = (205)(5.5)$$

F_m = 1,128 pounds per foot

(4) Find total force: Using Equation (5-46), find F_T:

$$F_T = F_s + F_m$$

EXAMPLE PROBLEM 33 (Continued)

$$F_T = 3{,}528 + 1{,}128$$

$$F_T = 4{,}656 \text{ pounds per foot}$$

e. Effect of Angle of Wave Approach. The dynamic forces due to breaking and broken waves can be reduced for those cases where the wave approaches at an angle relative to the wall (see Figure 124). The reduction is applied only to the dynamic force. The reduced dynamic force is:

$$F_{\alpha} = F \sin^2 \alpha \qquad (5\text{-}58)$$

WHERE: F_{α} = reduced dynamic component of force for breaking or broken wave striking structure at oblique angle

α = angle between axis of wall and direction of wave approach

F = dynamic component of force for breaking or broken wave if wall were perpendicular to direction of wave approach

This reduction is not applicable to nonbreaking waves nor to rubble-mound structures.

f. Nonvertical Walls. Dynamic forces can be reduced for breaking and broken waves impinging on walls that slope landward. The resultant dynamic force is:

$$f_{\theta} = F_{\alpha} \sin^2 \theta \qquad (5\text{-}59)$$

WHERE: F_{θ} = reduced horizontal dynamic component of force for breaking or broken wave striking nonvertical wall

F_{α} is obtained from Equation (5-58)

θ = angle between the horizontal and the structure slope

Recurved walls and stepped walls are treated as vertical walls.

2. UPLIFT FORCES.

a. General. Structures such as walls with overhanging members and piers should be designed so that the soffit of the deck is above the crest elevation of the maximum anticipated wave. If waves are expected to impinge on the deck, uplift forces may destroy the deck and its support. In the latter case, either the deck should be made strong enough to withstand the uplift forces, or vents or replaceable slabs should be installed to relieve the uplift forces. Special studies, such as those conducted by El Ghamry (1963), French (1969), and Wang (1967), should be undertaken for this complex design problem. Some uplift forces to be considered are those on wales, fenders, and overhanging decks generally associated with walls and piers. A preliminary method of estimating uplift forces on these objects, with dimensions which are much smaller than a wavelength, is given in the following subsection.

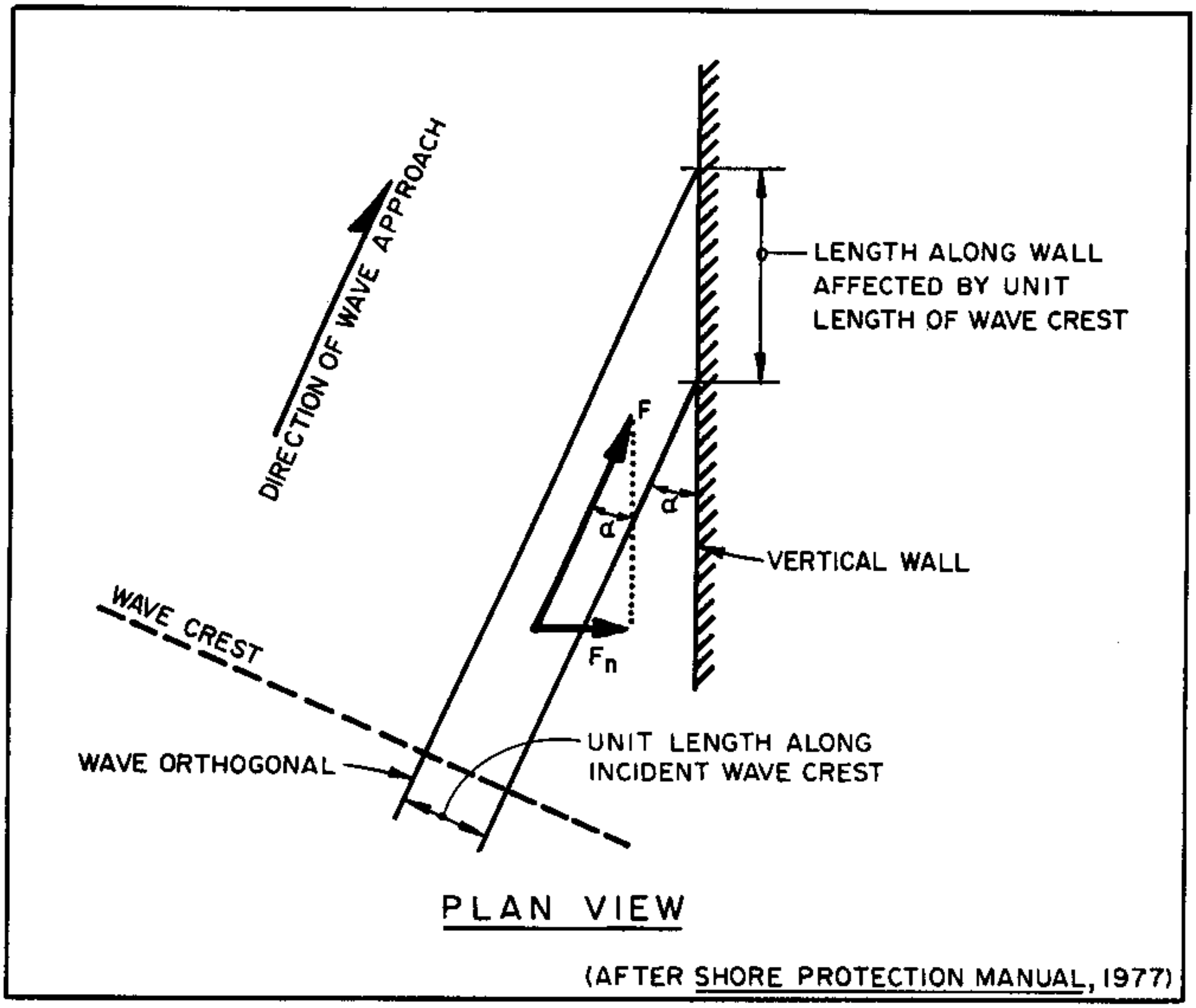


FIGURE 124
Diagram Showing Effect of Angle of Wave Approach

b. Forces on Wales. The vertical uplift force on a wale near a wall is assumed to be equal to or less than the hydrostatic pressure created by the elevation of the wave crest above the soffit of the wale times the projected area of the wale. Then the uplift force, F_u, is approximated by:

$$F_u = w_w (S_c - S_s) A \qquad (5\text{-}60)$$

WHERE: F = uplift force
w_w = unit weight of water
S = depth from clapotis crest (for a standing wave); determined by methods described in Section 5.1.b., Nonbreaking Waves (Equation (5-2))

S_s = depth from soffit to bottom
A = projected area of member

It may not be feasible to design for extreme design waves using this procedure. In the event of an extreme design wave occurring, damage may take place and wales or fenders would have to be repaired.

EXAMPLE PROBLEM 34

Given:
a. Incident wave height, H_i = 10 feet
b. Wave period, T = 6 seconds
c. Water depth, d = 40 feet
d. Depth from soffit to bottom, S = 45 feet
e. The wale is placed on the front of a smooth-faced wall and its dimension measured perpendicularly to the wall is 2 feet (see Figure 125).

Find: The wave force per unit length of wale.

Solution: (1) Find S_c:

$$\frac{H_i}{g T^2} = \frac{10}{(32.2) (6)^2} = 0.00863$$

$$\frac{H_i}{d_s} = \frac{10}{40} = 0.25$$

From Figure 85 for $H_i/g T^2 = 0.00863$ and $H_i/d_s = 0.25$:

$$\frac{h_o}{H_i} = 0.24$$

$h_o = (0.24) (10) = 2.4$ feet

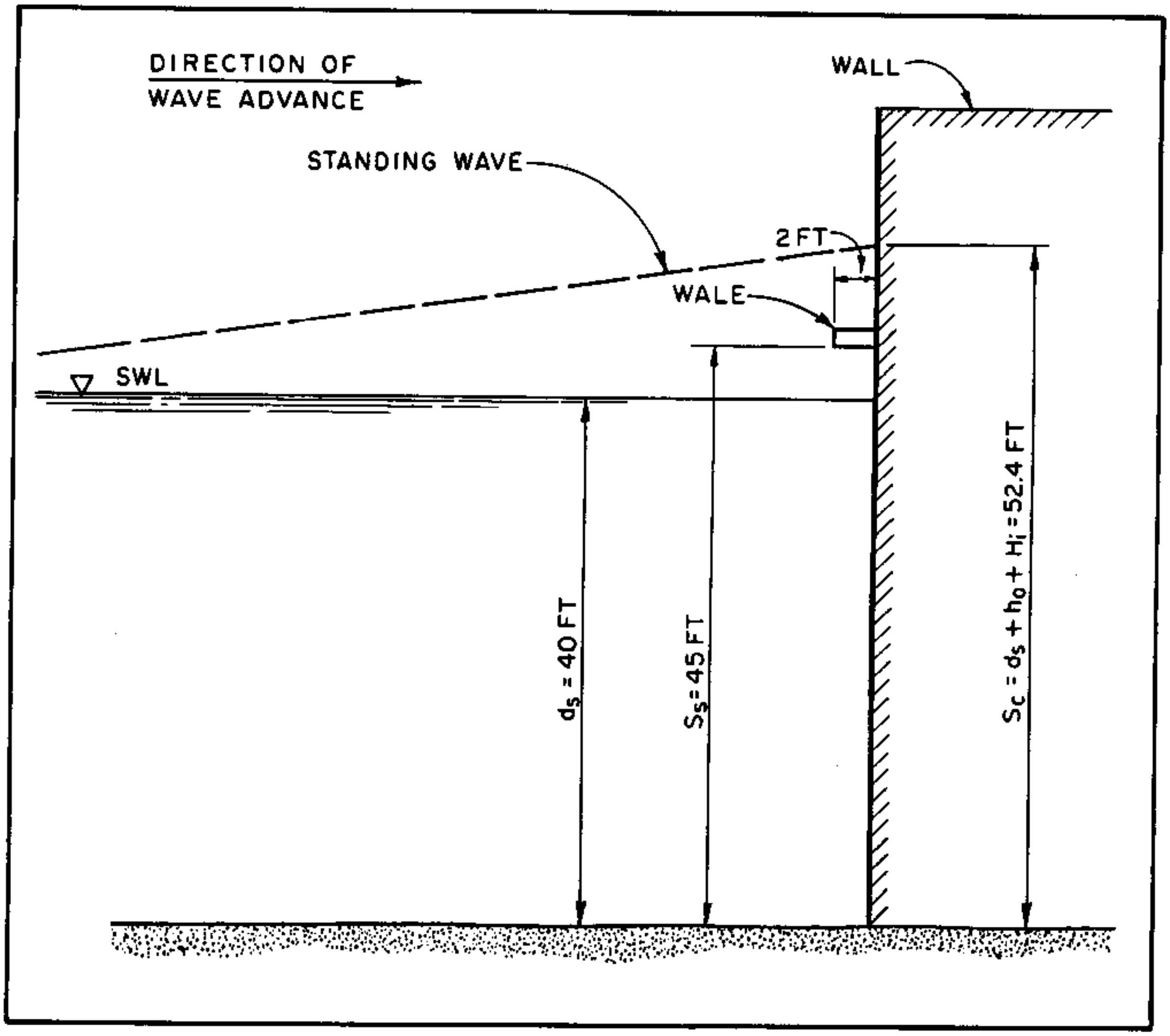


FIGURE 125
Diagram for Example Problem 34

EXAMPLE PROBLEM 34 (Continued)

Using Equation (5-2), find S_c:

$S_c = d_s + h_o + H_i$

$S_c = 40 + 2.4 + 10 = 52.4$ feet

(2) Using Equation (5-60), find F_u:

$F_u = w_w (S_c - S_s) A$

(Assuming salt water, $w_w = 64$ pounds per cubic foot.)

A = 2 square feet per foot of wale

$F_u = (64)(52.4 - 45)(2)$

$F_u = 947$ pounds per foot of wale

SECTION 6. FLOATING BREAKWATERS

1. DESCRIPTION. Floating breakwaters are a special classification of breakwater. A floating breakwater comprises a float, of sufficient size relative to the wavelength, held in place by mooring lines fixed to anchors or to guide piles. Figure 126 schematically shows a definition of terms. Figures 127 and 128 give schematic examples of several types of floating breakwater. Floating breakwaters can be constructed of barges, pontoons, floating docks, or rubber tires, or can be specially designed.

2. APPLICATION. Floating breakwaters are generally used or proposed for use in special cases, such as those in which: the water is deep, there is a large fluctuation of water level, the fetch is small (less than 2 to 3 miles), wind waves are less than 3 to 5 feet in height, and the period is short (generally less than 3 to 5 seconds). Generally, floating breakwaters are designed for temporary uses. For example, a floating breakwater may be used to provide temporary short period-wave protection during a dredging or construction project. Another possible application is to provide protection for a fleet landing or berthing area. Floating breakwaters can be easily transported compared to fixed breakwaters, and their installation is relatively insensitive to the subgrade conditions.

3. DESIGN PARAMETERS. The state-of-the-art of floating-breakwater design is in a relatively infant stage. Research has been conducted on several types of floating breakwaters, but there have been few long-term, successful installations. This section summarizes some general design principles. The transmission data can be used, with caution, for temporary installations. References are given for studies of special cases in Table 15.

a. Wave Transmission. The performance of a floating breakwater depends primarily upon the ratio of the structure beam, B, to the wavelength, L. Figure 129 plots the transmission coefficient, K_t, as a function of B/L for various moored structures; most of these structures are defined in Figures 127 and 128. The transmission coefficient, K_t, is equal to H_t/H_i, where H_t is the transmitted wave height and H_i is the incident wave height. The structure beam, B, should be on the order of half a wavelength to be assured of significant wave-height reduction. Figure 130 plots transmission coefficients versus B/L for a typical rubber-tire breakwater.

b. Mooring Forces. Mooring forces on and hardware for floating objects are discussed in DM-26.5 and DM-26.6. While the procedures given in DM-26.5 and DM-26.6 provide approximate forces for temporary moorings, more detailed studies are required for determining forces for permanent installations. Mooring forces depend not only on wave action and the type of structure, but also on the type of mooring. A pile-moored breakwater can have an order-of-magnitude greater mooring load than that for a breakwater moored by an anchor connected to chain or to a synthetic-fiber anchor line.

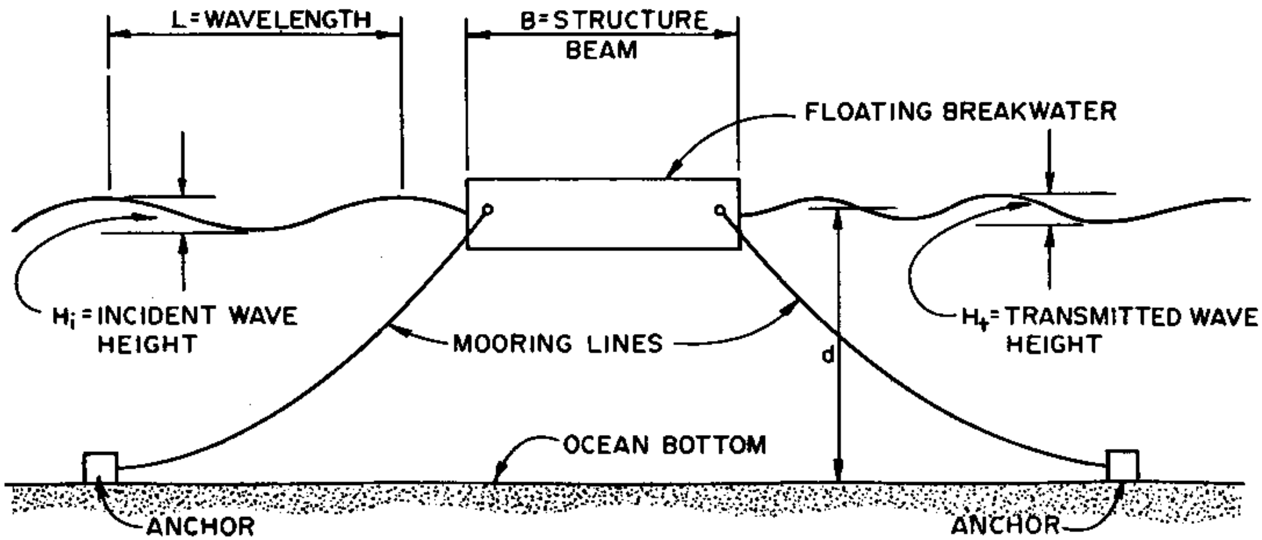


FIGURE 126
Definition of Floating-Breakwater Terms

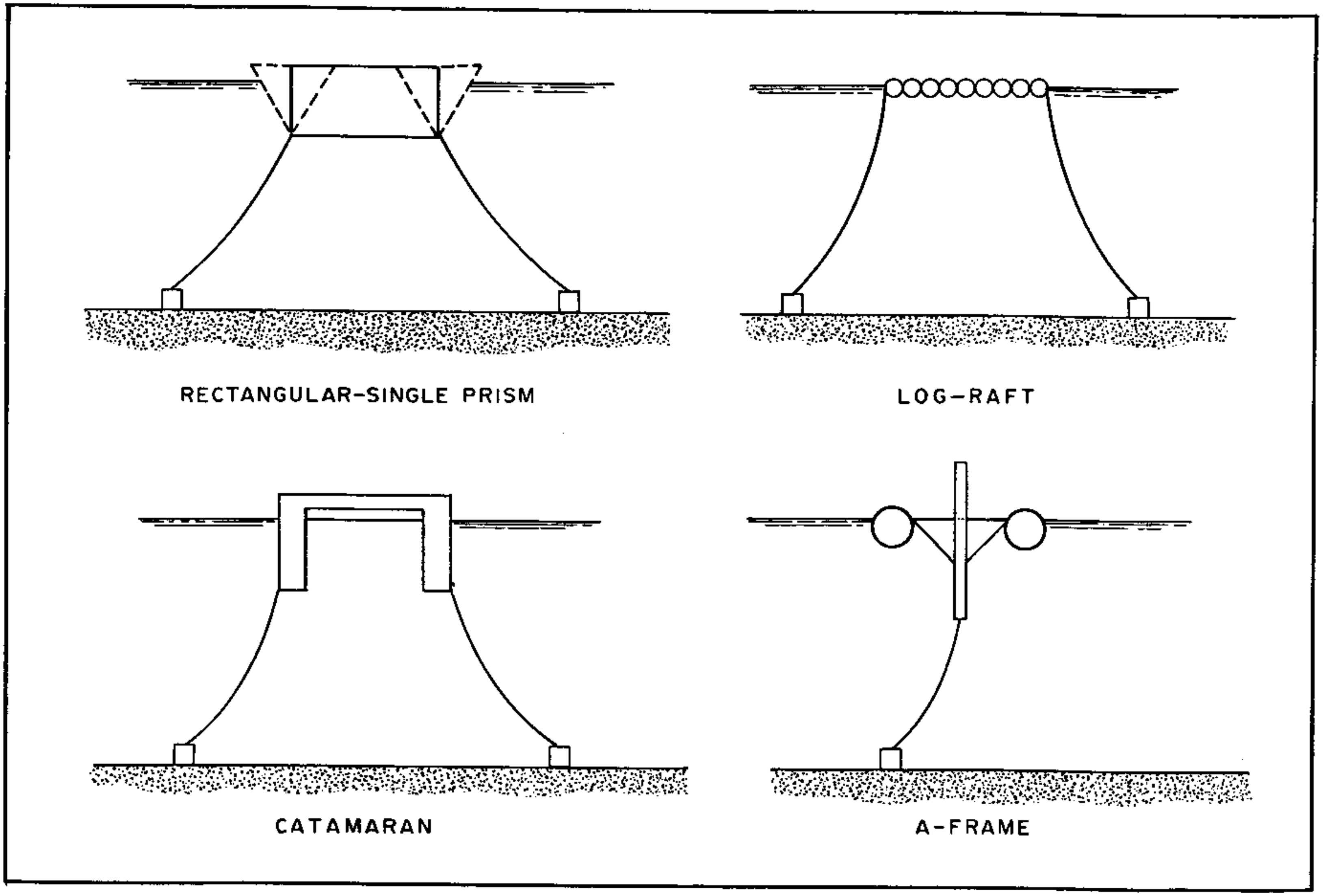


FIGURE 127
Four Examples of Floating-Breakwater Types

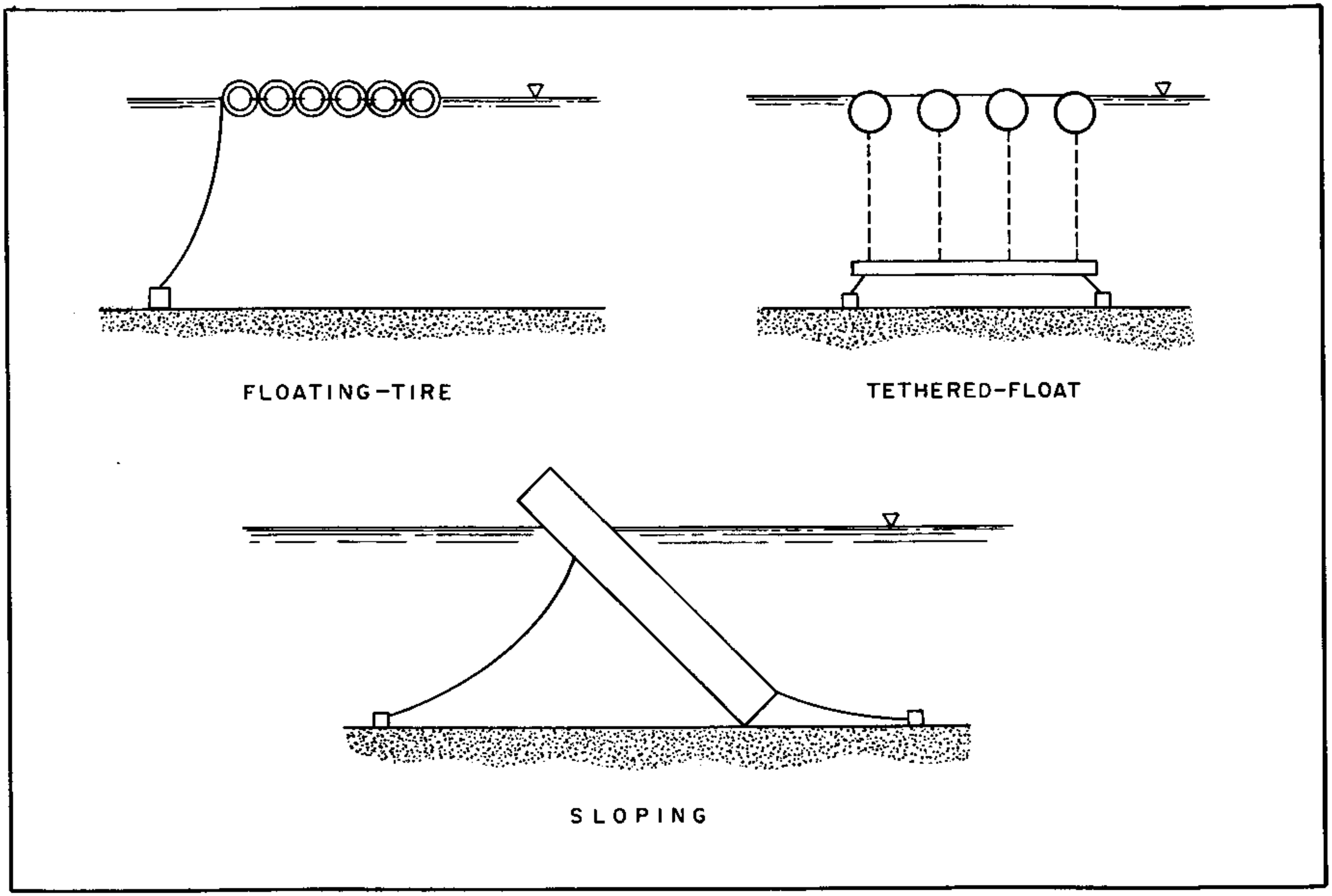


FIGURE 128
Three Examples of Floating-Breakwater Types

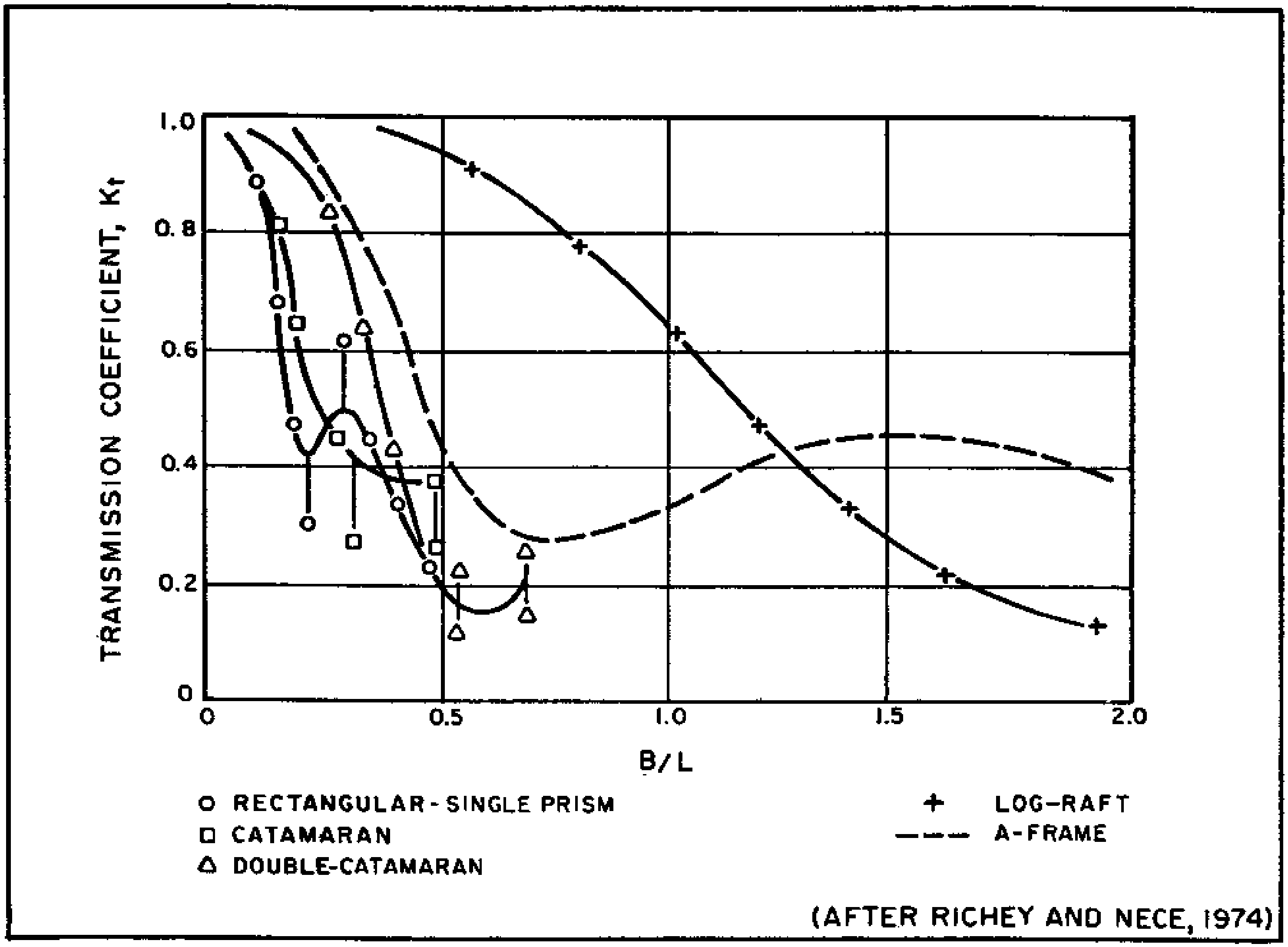


FIGURE 129
Transmission Coefficient Versus B/L for Various Moored Structures

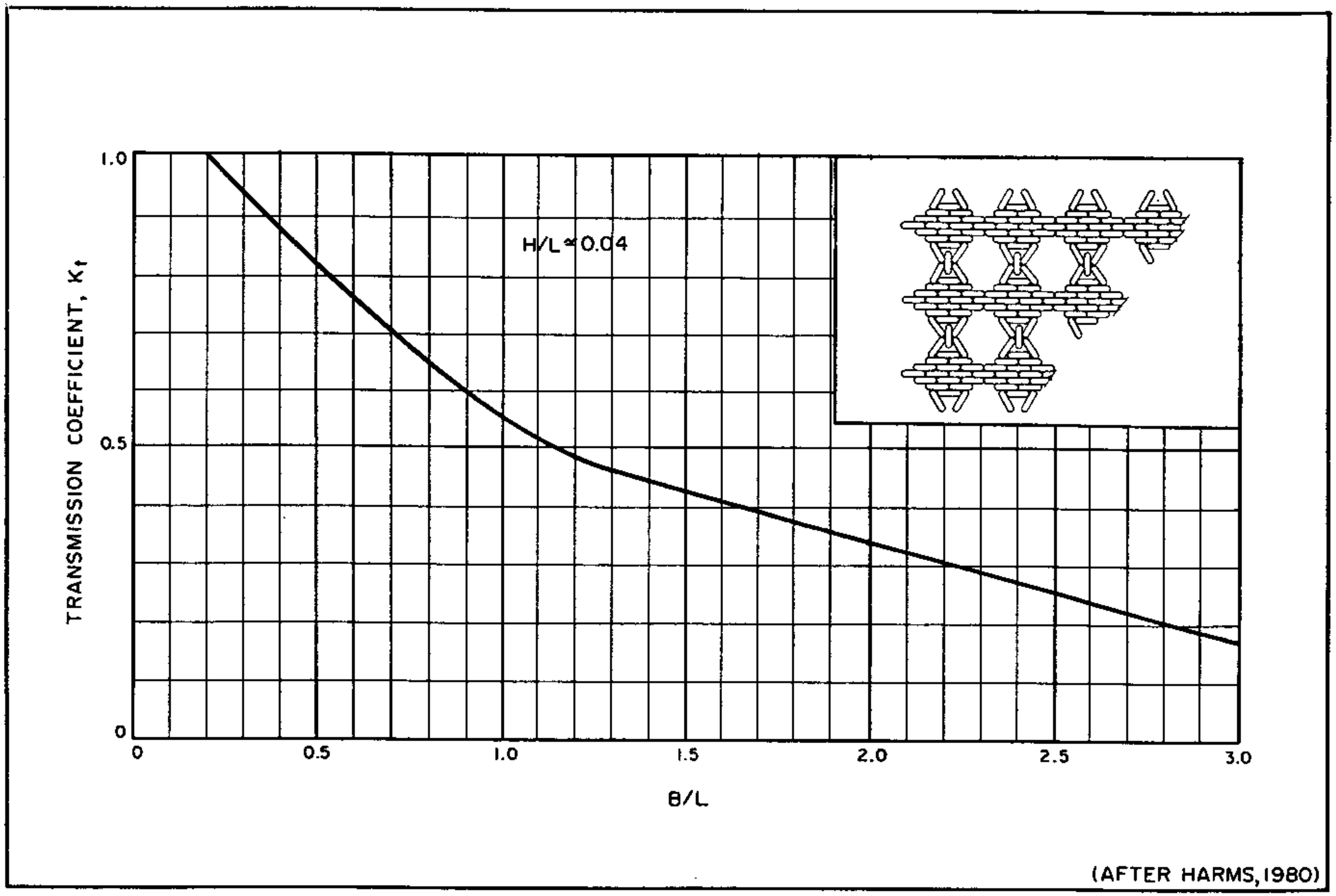


FIGURE 130
Transmission Coefficient Versus B/L for a Typical Rubber-Tire Breakwater

TABLE 15
References for Floating Breakwaters

Type of Structure	Reference
Floating breakwaters, general	Kowalski (1974) and Hales (1980)
Floating-tire, Wave-Guard	Harms (1980) Harms and Bender (1978)
Tethered-float	Seymour and Isaacs (1974)
Inclined-plane	Jones (1980)
Pontoon	Davidson (1971)

EXAMPLE PROBLEM 35

Given: a. Incident wave height, H_i = 3 feet
b. Water depth, d = 20 feet
c. Wave period, T = 3 seconds

Find: The necessary width, B, of a Goodyear floating-tire breakwater such that the transmitted wave height, H_t, is 1.5 feet.

Solution. (1) Find L:

$$L_o \ (g/2\pi) T^2 = (32.2/2\pi)(3)^2$$

$$L_o = 46.1 \text{ feet}$$

$$\frac{d}{L_o} = \frac{20}{46.1} = 0.434$$

From Figure 2 for $d/L_o = 0.434$:

$$\frac{d}{L} = 0.438$$

THEREFORE: $L_o = \frac{20}{0.438} = 45.7$ feet

(2) Desired $K_t = \frac{H_t}{H_i} = \frac{1.5}{3} = 0.5$

From Figure 130 for $K_t = 0.5$

B/L = 1.15

THEREFORE: B = 1.15 L = (1.15)(45.7) = 52.5 feet

4. METRIC EQUIVALENCE CHART. The following metric equivalents were developed in accordance with ASTM E-621. These units are listed in the sequence in which they appear in Section 6. Conversions are approximate.

2 miles = 3.2 kilometers
3 miles = 4.8 kilometers
3 feet = 91.4 centimeters
5 feet = 1.5 meters

SECTION 7. WAVE FORCES ON CYLINDRICAL PILES

1. INTRODUCTION. Methods for the calculation of wave forces on piles are divided into eight cases, as listed in Table 16. The cases are examined in order of complexity. The relative complexity depends upon:

(1) The shape of the pile: piles may be either uniform or nonuniform in diameter from the bottom to the surface. Nonuniformity in diameter is usually a result of marine fouling near the intertidal zone or of the use of a protective jacket around a section of the pile.

(2) The complexity of the structure: force may be calculated for a single pile or for a group of piles.

(3) The ratio of the pile diameter to the local wavelength and to the local wave height: these ratios determine the relative effects of inertial forces and wave diffraction.

(4) The accuracy needed for the project: preliminary planning studies may only require an order of magnitude estimate of the wave force.

Case 1 assumes a uniform pile of small diameter compared to the local wavelength and wave height. Calculations of the wave force are based only on the drag force. Case 2 assumes a uniform pile of intermediate diameter compared to the local wavelength and wave height. Calculations of the wave force are based on both the inertial and drag forces. Case 3 provides a method for determining the wave forces on a pile of small, nonuniform diameter. The assumptions made are the same as for Case 1 and the estimates of wave force are based solely on drag. The method uses linear theory but provides a nonlinear correction factor. Case 4 provides an approximate method for calculating the force-time history on a single pile of nonuniform, intermediate diameter. This method incorporates both inertial and drag forces; however, it is based on linear theory and, therefore, should be used only for preliminary design. Case 5 provides nonlinear corrections which, when used in conjunction with the method outlined in Case 4, will provide wave forces for final design. Case 6 provides a method for determining wave forces on a combination of piles. Case 7 provides a method for determining the wave forces on bracing used between vertical piles. Case 8 outlines a method for determining the breaking-wave forces on a pile.

2. BASIC EQUATIONS. Figure 131 defines terms used to describe wave forces on piles. Basic equations and other definitions required for calculation of wave-induced forces on cylindrical piles are as given below.

a. Forces. The total force, F_T, on a pile subjected to nonbreaking waves is the sum of the drag force, F_D and the inertial force, F_I:

$$F_T = F_D + F_I \qquad (7\text{-}1)$$

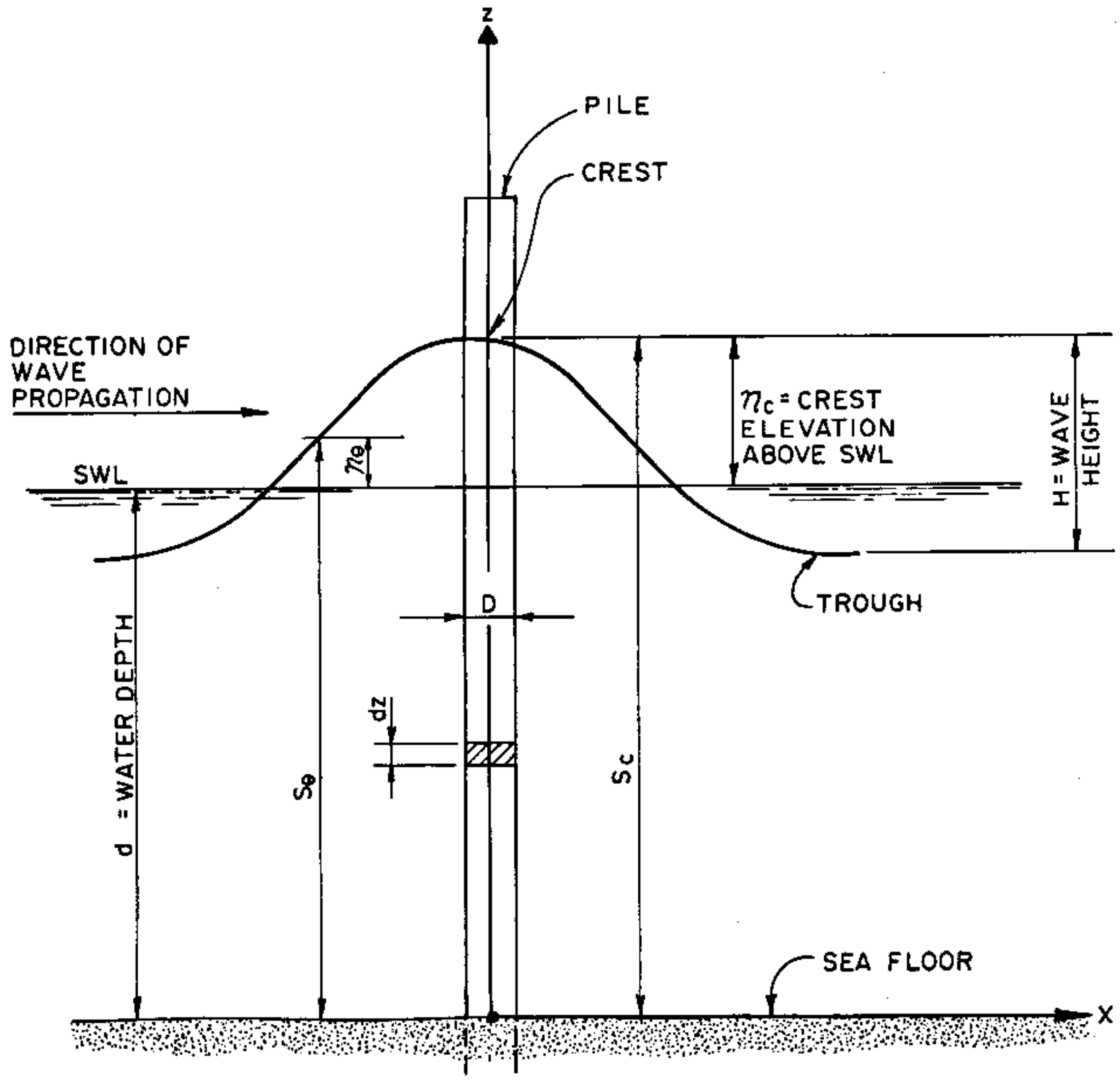
The drag force is given by:

$$F_D = \int_o^z f_D \, dz \qquad (7\text{-}2)$$

WHERE: z = depth in terms of vertical distance along a coordinate axis with its origin at the bottom

TABLE 16
Wave Forces on Piles

Case	Description	Application	Section	Parameters
Uniform Diameter Only				
1	Drag only	Preliminary design: small-diameter piles	7.5	d/L_o, H/H_b
2	Drag and inertial—maximum nonlinear theory	Preliminary design: intermediate-diameter piles	7.6	d/L_o, H/H_b, $(C_M D)/(C_D H)$
Nonuniform and Uniform Diameters				
3	Drag	Preliminary design: small-diameter piles	7.7	d/L_o, z/d
4	Drag and inertial	Preliminary design: intermediate-diameter piles, arbitrary wave-phase angle	7.8	d/L_o, z/d, $(C_M D)/(C_D H)$
5	Nonlinear corrections	Final design for Case 4: arbitrary wave-phase angle	7.9	d/L_o, z/d, $(C_M D)/(C_D H)$, θ, H/H_b
Miscellaneous				
6	Combination of piles	Bents	7.10	All of the above, plus phase difference between piles
7	Bracing	Crossmembers	7.11	
8	Forces due to breaking waves	Shallow water	7.12	



NOTE: IN SECTION 7, THE NOTATION USED FOR THE FREE SURFACE WILL BE AS FOLLOWS:

η_θ = VALUE OF FREE-SURFACE ELEVATION FROM SWL AT AN ARBITRARY WAVE-PHASE ANGLE, θ

η_c = VALUE OF FREE-SURFACE ELEVATION FROM SWL (MEASURED AT THE PILE) WHEN THE WAVE CREST IS AT THE PILE

S_θ = DISTANCE OF FREE SURFACE MEASURED FROM THE BOTTOM AT AN ARBITRARY WAVE-PHASE ANGLE, θ

S_c = DISTANCE OF FREE SURFACE FROM THE BOTTOM (MEASURED AT THE PILE) WHEN THE WAVE CREST IS AT THE PILE

FIGURE 131
Definition of Terms Used to Describe Wave Forces on Piles

$$f_D = \left(\frac{1}{2}\right) [\rho] C_D D |u| u \qquad (7\text{-}3)$$

= drag force per unit length of pile

[rho] = density of water

C_D = drag coefficient (dimensionless), obtained from Table 17

D = pile diameter

u = instantaneous horizontal water-particle velocity

TABLE 17

Drag Coefficient

C_D	Reynolds Number, R_e	Comments
0.7	> 5 x 10^5	Used for most design applications
1.2 to 0.7	2 x 10^5 to 5 to 10^5	Traditional
1.2	< 2 x 10^5	Subcritical

The inertial force, F_I, is given by:

$$F_I = \int_o^z f_I \, dz \qquad (7\text{-}4)$$

WHERE: $f_I = \rho C_M \pi (D^2/4)(du/dt)$ **(7-5)**

= inertial force per unit length of pile

C_M **= inertial, or added-mass, coefficient (dimensionless), obtained from Table 18**

du/dt = instantaneous horizontal water-particle acceleration

TABLE 18

Inertia Coefficient

C_M	Reynolds Number, R_e	Comments
2.0	> 2.5 x 10^5	

1.5 to 2.0	2.5 x 10À5Ù to 5 to 10À5Ù	
1.5	< 5 x 10 À5Ù	Used for most design applications

A transverse force due to alternate eddy formation and shedding, known as the "lift force," will also result from the wave-pile interaction. The lift force acts perpendicularly to both wave direction and pile axis. The lift force is analogous to a drag force and is maximum at the same moment as the drag force. For design of rigid structures, a lift force is automatically taken into account by adopting a conservative CD value regardless of the force direction. For design of flexible structures, dynamic response of the structure must be analyzed (see Laird (1962)).

b. Moments. The moment, M, about the mudline is given by:

$$M = \int_{o}^{z} (f_D + f_I)\, z\, dz \qquad (7\text{-}6)$$

WHERE: z = depth in terms of vertical distance along a coordinate axis with its origin at the bottom

f_D = drag force per unit length of pile (Equation (7-3))

f_I = inertial force per unit length of pile (Equation (7-5))

c. Drag and Inertial Coefficients. Drag and inertial coefficients for cylindrical piles subject to ocean waves are a function of Reynolds number. The Reynolds number, RÚe¿, is given by:

RÚe¿ = (uÚm¿D)/[nu] (7-7)

WHERE: uÚm¿ = approximate maximum horizontal water-particle velocity given by the appropriate equation and [theta] = 0 deg. for u in Table 1 (Section 1) at z = [eta]Úc¿. Note that the value of z is referenced to SWL in Section 1 instead of to the bottom, as in Section 7. (See Table 1 for Water Particle Velocity, Horizontal.)

[theta] = wave-phase angle

z = depth in terms of vertical distance along a coordinate axis with its origin at the bottom

[eta]Úc¿ = water-surface elevation at wave crest relative to SWL

H = local wave height

g = gravitational acceleration (432.2 feet per secondÀ2Ù)

d = water depth

D = pile diameter

[nu] = kinematic viscosity (approximately 1 x 10 feet per second for salt water)

3. LIMITING WAVE HEIGHT, The linear and nonlinear stream-function wave theories (Dean, 1965) are used for calculation of wave forces. The forces are calculated throughout Section 7 by application of a series of figures and equations which relate the given wave height to the limiting height over a horizontal bottom. Calculation requires entering the figures with a value of H/H_b, where H_b = breaking-wave height, to find various force and moment coefficients. The local wave height, H, can be determined through use of the nonlinear shoaling curve (Figure 4) for a nonhorizontal bottom, For horizontal bottoms, the local wave height, H, is determined using the linear shoaling coefficient (Figure 2). The breaking-wave height, H_b, can be determined using Figure 42 for nonhorizontal bottoms. H_b is taken as 0.78 d for horizontal bottoms.

Stream-function theory has been developed strictly for a horizontal bottom; however, sloped bottoms occur more frequently in design situations. Consequently, while only strictly accurate for a horizontal bottom, the methods presented herein, in conjunction with the aforementioned methods for determining local wave heights over a sloped bottom, should provide conservative estimates of wave forces on piles. If H/H_b is found to be > 1, refer to Subsection 7.12, CASE 8--FORCES DUE TO BREAKING WAVES. Example Problem 36 demonstrates the determination of H/H_b.

EXAMPLE PROBLEM 36

Given:
a. Equivalent unrefracted deepwater wave height, H'_o = 10 feet
b. Water depth, d = 17 feet
c. Wave period, T = 10 seconds
d. Bottom slope, m = 0.02

Find: H_b and H/H_b for wave-force calculations.

Solution: (1) Find H:

(a) Calculate L_o, d/L_o, and H'/L_o:

$L_o = (g/2\pi) T^2 = (32.2/2\pi)(10)^2 = 512$ feet

$d/L_o = 17/512 = 0.0332$

$H'_o/L_o = 10/512 = 0.0195$

(b) From Figure 4 for $d/L_o = 0.0332$ and $H'_o/L_o = 0.0195$:

$H/H'_o = 1.13$

$H = 1.13\ H'_o$

$H = (1.13)(10) = 11.3$ feet

(2) Find H_b:

EXAMPLE PROBLEM 36 (Continued)

(a) Find H'_o/gT^2:

$$\frac{H'_o}{gT^2} = \frac{10}{(32.2)(10)^2} = 0.00311$$

(b) From Figure 42 for $H'_o/gT^2 = 0.00311$ and $m = 0.02$:

$$\frac{H_b}{H'_o} = 1.2$$

$$H_b = (1.2)(10) = 12 \text{ feet}$$

(3) Find H/H_b:

$$\frac{H}{H_b} = \frac{11.3}{12}$$

$H/H_b = 0.942$; use $H/H_b = 0.94$

4. WAVE-CREST ELEVATION. Knowledge of the wave-crest elevation above still water level, η_c, is necessary not only to calculate wave force, but also to determine the height of a pile-supported structure above sea level. The relative wave-crest elevation above the still water level, η_c/H, is greater than or equal to one-half ($\eta_c/H \geq 1/2$) and is given in Figure 132 as a function of d/gT^2 and H/gT^2. Figure 132 is based on stream-function theory and was developed by Seelig and Ahrens (1981). The distance of the wave crest above the bottom when the crest is at the pile, S_c, is:

$$S_c = \eta_c + d \qquad (7\text{-}8)$$

WHERE: S_c = distance of free surface measured from the bottom to the wave crest when the crest is at the pile

η_c = water-surface elevation at wave crest relative to SWL

d = water depth at the pile from SWL

Example Problem 37 shows the calculation of S_c.

EXAMPLE PROBLEM 37

Given:
a. Equivalent unrefracted deepwater wave height, H'_o = 10 feet
b. Water depth, d = 17 feet
c. Wave period, T = 10 seconds

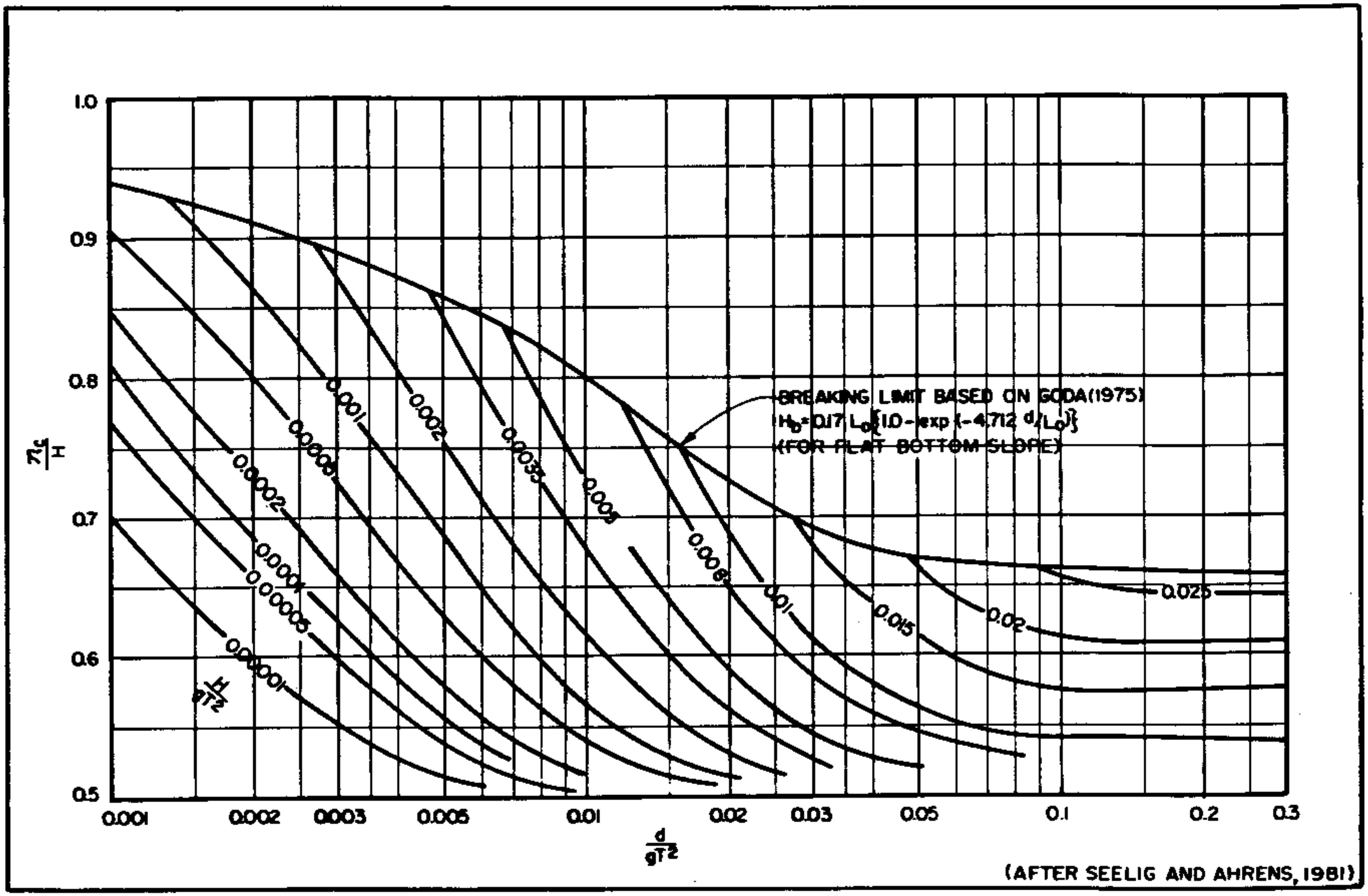


FIGURE 132
Relative Wave-Crest Elevation Above SWL, η_c/H, as a Function of d/gT^2 and H/gT^2

a Function of d/g TÀ2Ù and H/g TÀ2Ù

26.2-244

EXAMPLE PROBLEM 37 (Continued)

Find: Wave-crest elevation above the bottom, S_c.

Solution: (1) From Example Problem 36, H = 11.3 feet

(2) Find d/gT^2 and H/gT^2:

$$\frac{d}{gT^2} = \frac{17}{(32.2)(10)^2} = 0.00528$$

$$\frac{H}{gT^2} = \frac{11.3}{(32.2)(10)^2} = 0.00351$$

(3) From Figure 132 for $d/gT^2 = 0.00528$ and $H/gT^2 =$ 0.00351:

$\eta_c/H = 0.83$

$\eta_c = (0.83)(11.3) = 9.38$ feet; use $\eta_c = 9.5$ feet

$S_c = \eta_c + d = 9.5 + 17 = 26.5$ feet

5. CASE 1--MAXIMUM FORCE ON SINGLE PILE OF SMALL, UNIFORM DIAMETER (PRELIMINARY DESIGN).

a. Range of Application. Case 1 is applicable to piles of small, uniform diameter and for values of D/H and d/L_o as follows:

(1) Shallow and transitional water: Case 1 is applicable in shallow and transitional water when $d/L_o < 0.1$ and $D/H < 0.6$.

(2) Deep water: Case 1 is applicable in deep water when $d/L_o > 0.5$ and $D/H < 0.15$.

(3) If the conditions in (1) or (2) are not met, refer to Case 2.

Note: See Section 1, Table 1, for a definition of shallow, transitional, and deep water.

b. Maximum Drag Force. The maximum drag force, F_{mD}, occurs under the crest and is determined by:

$$F_{mD} = \left(\frac{1}{2}\right) \rho C_D D \left(\frac{H}{T}\right)^2 d \, (K_{DM} \phi_D) \qquad (7\text{-}9)$$

WHERE: ρ = density of water

C_D = drag coefficient (obtained from Table 17)

D = pile diameter

H = local wave height

T = wave period

d = water depth

The value of (K_{DM} ϕ_D) can be obtained from Figure 133 as a function of d/L_o and H/H_b.

c. Maximum Moment. The maximum drag moment, M_{mD} is given by:

$$M_{mD} = \left(\frac{1}{2}\right) \rho C_D D \left(\frac{H}{T}\right)^2 d^2 (\tau_{DM} \psi_D) \qquad (7\text{-}10)$$

WHERE: ρ = density of water

C_D = drag coefficient (obtained from Table 17)

D = pile diameter

H = local wave height

T = wave period

d = water depth

The value of (τ_{DM} ψ_D) can be obtained from Figure 134 as a function of d/L_o and H/H_b.

d. Reaction. The lever arm, or distance of the point of application of force above the bottom, z_{mD}, is:

$$z_{mD} = \frac{M_{mD}}{F_{mD}} \qquad (7\text{-}11)$$

EXAMPLE PROBLEM 38

Given:
a. Equivalent unrefracted deepwater wave height, H'_o = 10 feet
b. Water depth, d = 17 feet
c. Wave period, T = 10 seconds
d. Vertical pile with diameter, D = 1 foot
e. Bottom slope, m = 0.02

Find: The maximum force and moment about the mudline on the vertical pile; also find the lever arm above the bottom.

Solution: (1) From Example Problem 36:

$d/L_o = 0.0332$

H = 11.3 feet, H_b = 12 feet, and $H/H_b = 0.94$

(2) From Example Problem 37:

$d/g T^2 = 0.00528$

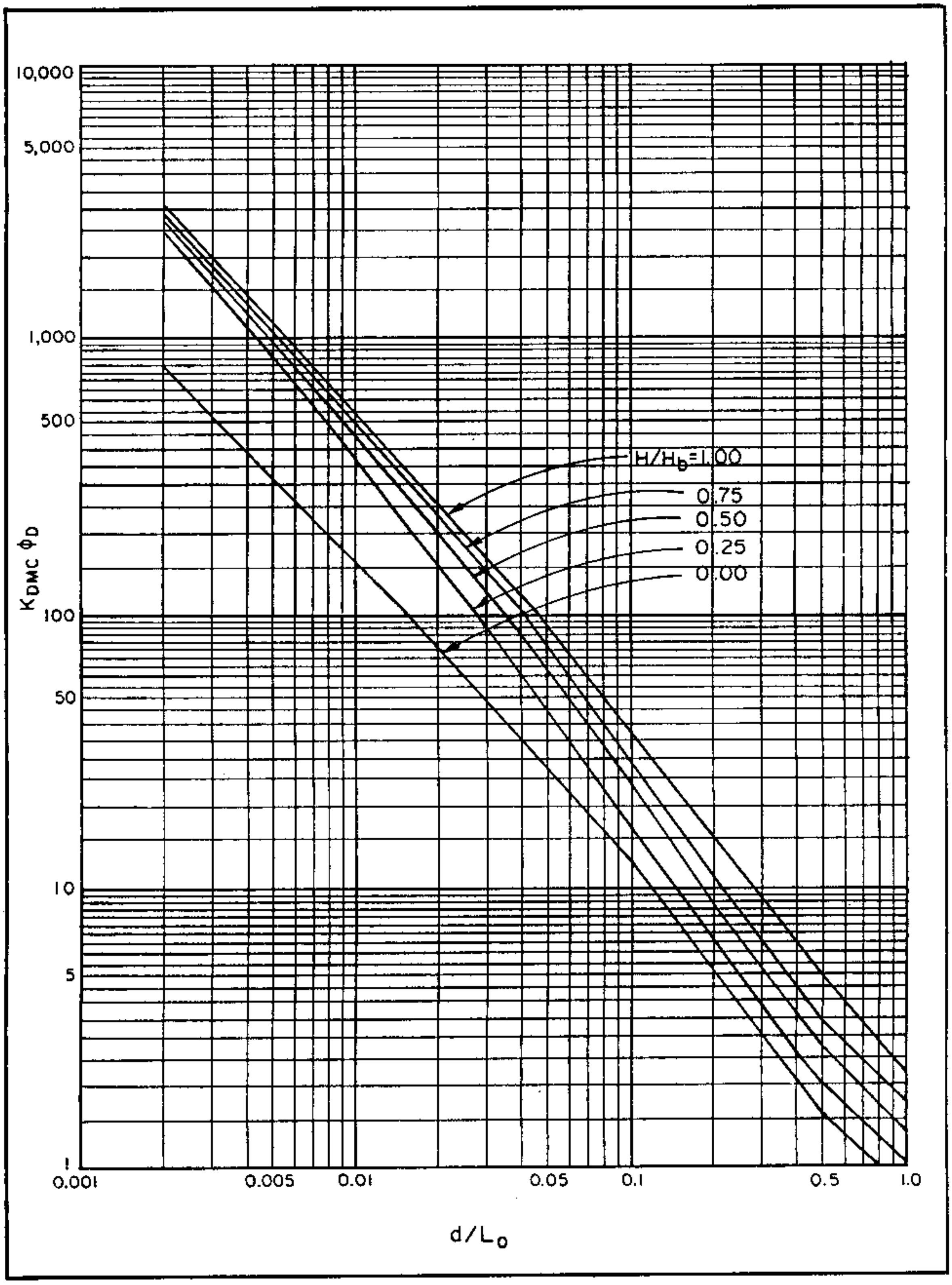


FIGURE 133

$(K_{DMC}\,\Phi_D)$ as a Function of d/L_o and H/H_b

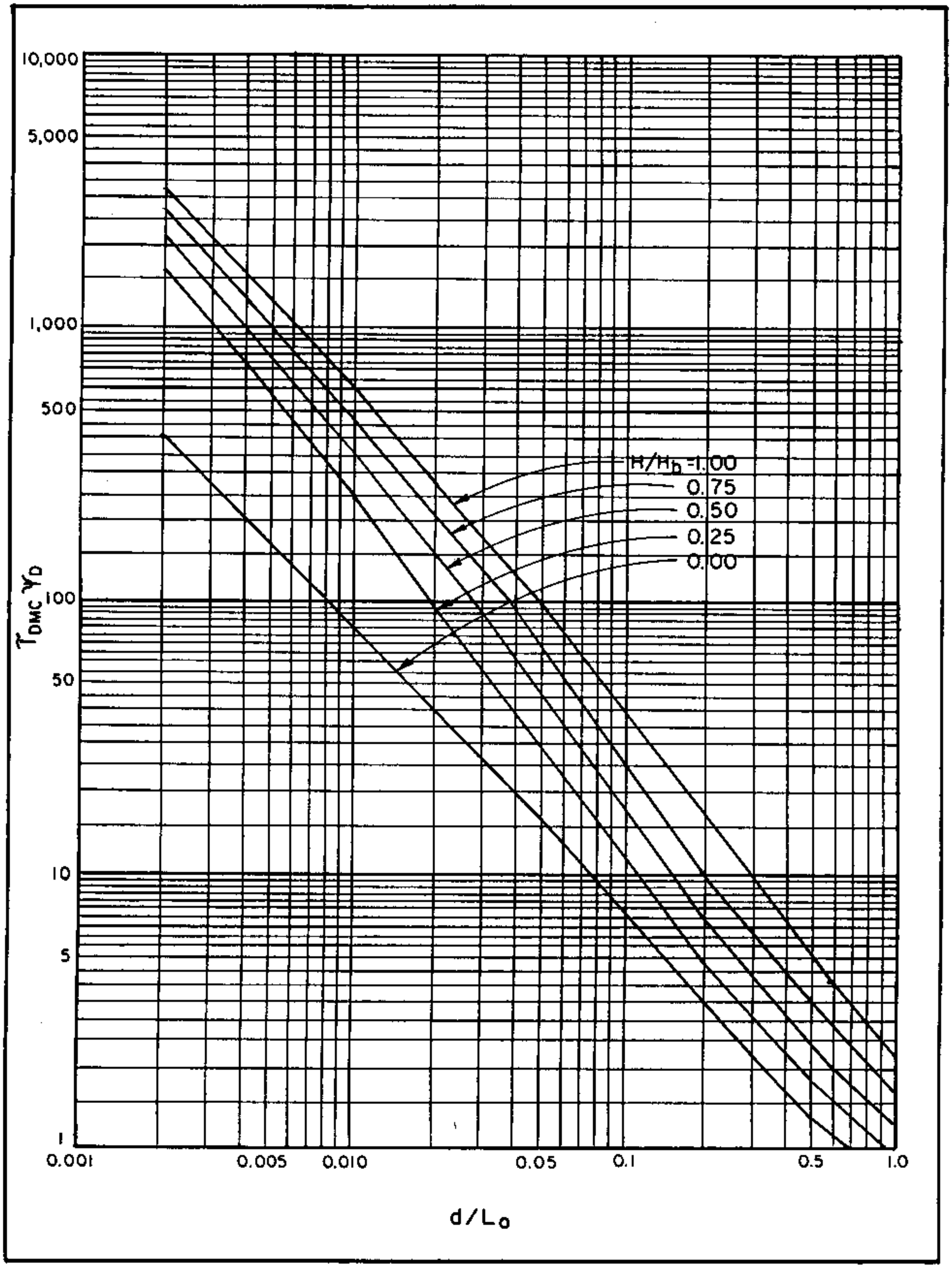


FIGURE 134

(τ_{DMC} Ψ_D) as a Function of d/L_o and H/H_b

EXAMPLE PROBLEM 38 (Continued)

$H/gT^2 = 0.00351$

$\eta_c = 9.5$ feet

(3) Determine if Case 1 is applicable to this problem:

(a) Determine if water depth is shallow, transitional, or deep:

From Figure 2 for $d/L_o = 0.0332$:

$d/L_o = 0.075$

$$\frac{1}{25} < \frac{d}{L} \quad \frac{1}{2}$$

The wave is in transitional water.

(b) $d/L_o = 0.0332$ (from Example Problem 36)

(c) Find D/H:

$$\frac{D}{H} = \frac{1}{11.3} = 0.088$$

$d/L_o = 0.0332 < 0.1$

and $D/H = 0.088 < 0.6$

THEREFORE: Case 1 is applicable.

(4) Determine Reynolds number, R_e:

(a) Find L:

$$\frac{d}{L} = 0.075$$

$$L = \frac{d}{0.075}$$

$$L = \frac{17}{0.075} = 227 \text{ feet}$$

(b) Find u_m: From Table 1 (Section 1) for transitional water; $u = u_m$:

(Note that in this equation, z is referenced to SWL.)

$$H \qquad gT \qquad \cosh[2\pi(z + d)/L]$$

uÚm¿ = (Ä) (ÄÄ) (ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ(cos[theta])
cosh(2[pi] d/L

For the maximum value of u, use zÚc¿ = [eta]Úc¿ = 9.5 feet and cos [theta] = 1

EXAMPLE PROBLEM 38 (Continued)

$$u_m = \left(\frac{11.3}{2}\right) \left[\frac{(32.2)(10)}{(227)}\right] \left\{\frac{\cosh [2\pi (9.5 + 17)/227]}{\cosh[2\pi (17/227)]}\right\}(1)$$

$$u_m = 8.01 \left[\frac{\cosh (0.733)}{\cosh (0.471)} \right]$$

From Figure 3:

$\cosh (0.733) = 1.28$

and $\cosh (0.471) = 1.11$

$$u_m = (8.01) \left(\frac{1.28}{1.11} \right) = 9.2 \text{ feet per second}$$

(c) Using Equation (7-7), find R_e:

$R_e = (u_m D) / \nu$

$R_e = [(9.2)(1)] / (1 \times 10^{-5})$

$R_e = 9.2 \times 10^5$

(5) Find F_{mD}:

(a) Determine drag coefficient, C_D:

From Table 17 for $R_e = 9.2 \times 10^5$:

$C_D = 0.7$

(b) Determine $(K_{DMC} \, \phi_D)$:

From Figure 133 for $d/L_o = 0.0332$
and $H/H_b = 0.94$

$(K_{DMC} (\phi_D)) = 154$

(c) Assuming salt water, $\rho = 2$ slugs per cubic foot.

(d) Using Equation (7-9), the maximum force is:

$$F_{mD} = \left(\frac{1}{2}\right) \rho \, C_D \, D \left(\frac{H}{T}\right) d \, (K_{DMC} \, \phi_D)$$

$$F_{mD} = \left(\frac{1}{2}\right) (2) (0.7) (1) \left(\frac{11.3}{10}\right)^2 (17)(154)$$

$F_{mD} = 2{,}340$ pounds

(6) Find MÚmD¿:

(a) Determine ([tau]ÚDMC¿ [psi]ÚD¿):

From Figure 134 for d/LÚo¿ = 0.0332 and H/HÚb¿ = 0.94:

EXAMPLE PROBLEM 38 (Continued)

$([\tau]_{DMC}\ [\psi] = 147$

(b) Using Equation (7-10), the maximum moment is:

$$M_{mD} = \left(\frac{1}{2}\right) \rho\, C_D \left(\ \right)^2 d^2 \left([\tau]_{DMC}\ [\psi]_D\right)$$

$$M_{mD} = \left(\frac{1}{2}\right)(2)\ (0.7)\ (1)\ \left(\frac{11.3}{10}\right)^2\ (17)^2\ (147)$$

M_{mD} = 37,973 foot-pounds

(7) Find z_{mD}:

Using Equation (7-11), the lever arm above the bottom is:

$$z_{mD} = \frac{M_{mD}}{f_{mD}} = \frac{37,973}{2,340}$$

z_{mD} = 16.2 feet

6. CASE 2--PILE OF INTERMEDIATE, UNIFORM DIAMETER (PRELIMINARY DESIGN).

a. Range of Application. This method is applicable to most cases of H/D and d/L_o, although the pile diameter should be small compared to the wavelength (that is, $D/L_o < 0.2$). For larger values of D/L, the diffraction effects induced by the wave interacting with a pile of larger diameter cannot be neglected. Case 2 is strictly applicable to single piles where the spacing between piles, δ, is greater than 10 D ($\delta > 10$ D). However, reasonable accuracy can be obtained using this method for closer-spaced piles to an arbitrary limit, $\delta > 2$ D.

b. Maximum Force. The maximum force on a single pile is the sum of drag and inertial forces. This maximum force occurs in front of the wave crest. The maximum force, F_{mDI}, is given by:

$$F_{mDI} = \rho\, g\, C_D\, H^2\, D\, \phi_m \qquad (7\text{-}12)$$

WHERE: ρ = density of water

g = gravitational acceleration (32.2 feet per second^2)

C_D = drag coefficient (obtained from Table 17)

H = local wave height

D = pile diameter

ϕ_m = a coefficient given in Figures 135 through 138 as a function of d/gT^2 and H/gT^2 for various values of W

```
         CÚM¿ D
W      = ÄÄÄÄÄÄ = parameter used in pile-force and moment          (7-13)
         CÚD¿ H   calculations
```

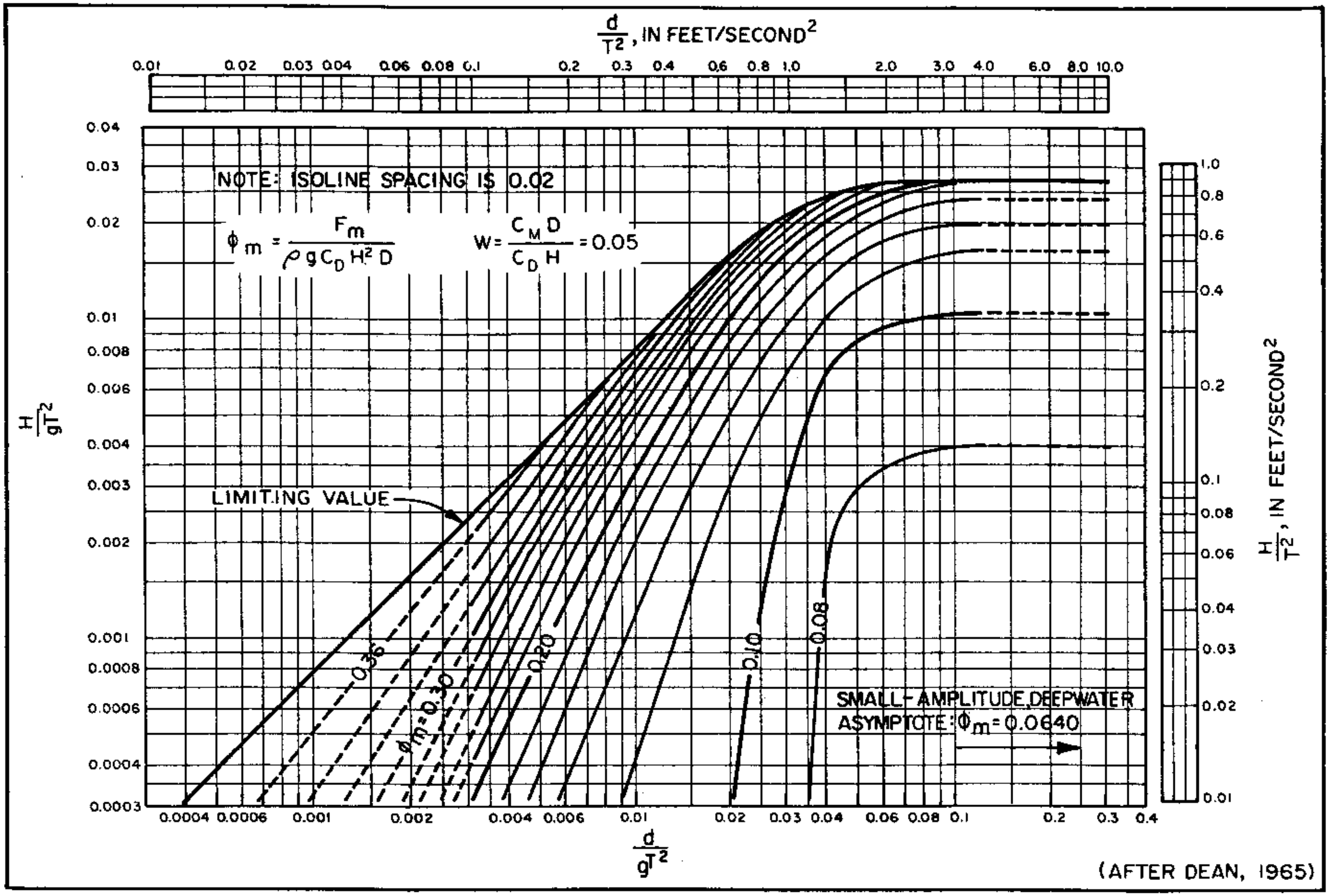


FIGURE 135
Isolines of ϕ_m as a Function of $d/g\,T^2$ and $H/g\,T^2$ for $W = 0.05$

TÀ2Ù for W = 0.05]

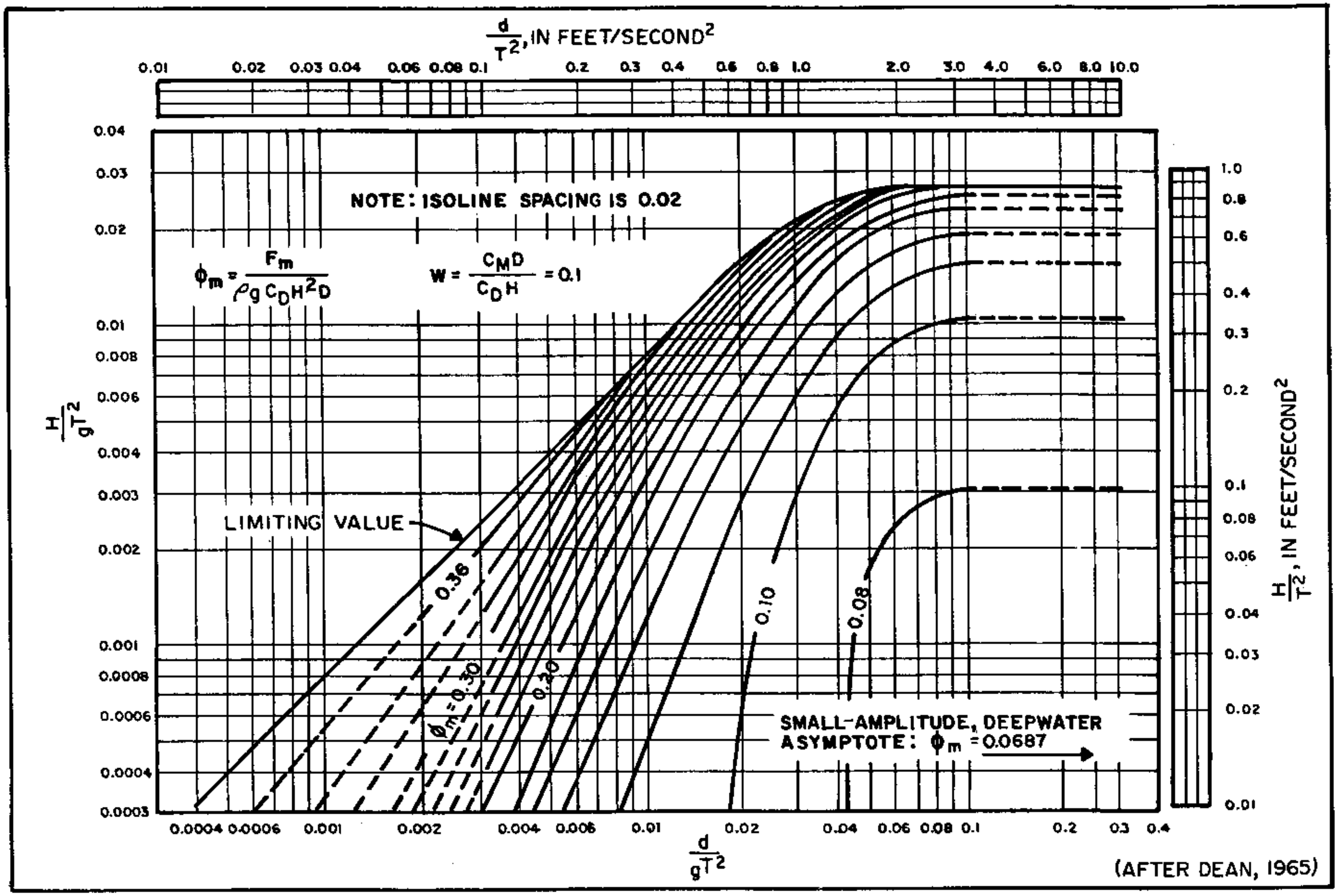


FIGURE 136

Isolines of ϕ_m as a Function of $d/g\,T^2$ and $H/g\,T^2$ for $W = 0.1$

TÀ2Ù for W = 0.1]

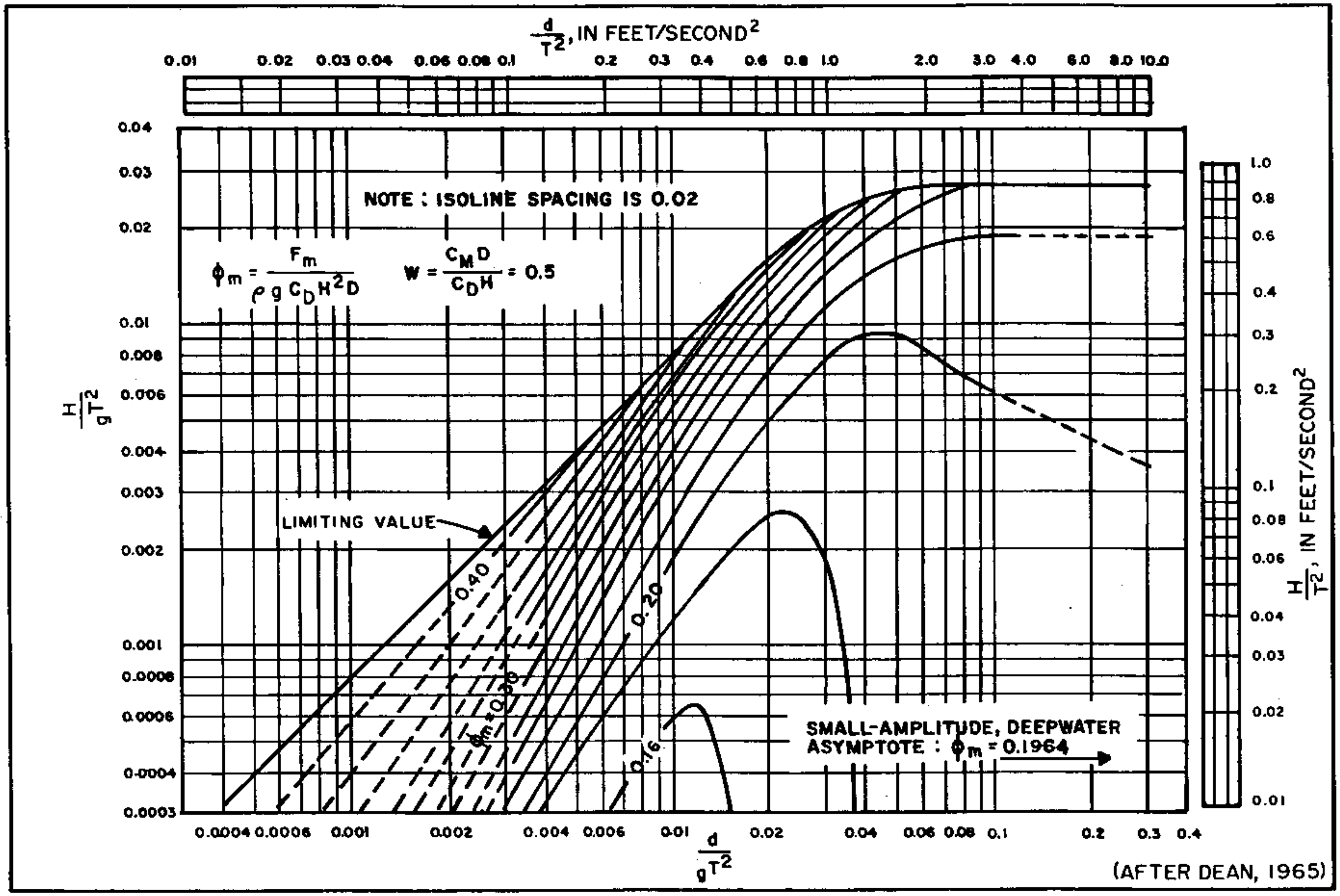


FIGURE 137
Isolines of ϕ_m as a Function of $d/g\,T^2$ and $H/g\,T^2$ for $W = 0.5$

TÀ2Ù for W = 0.5]

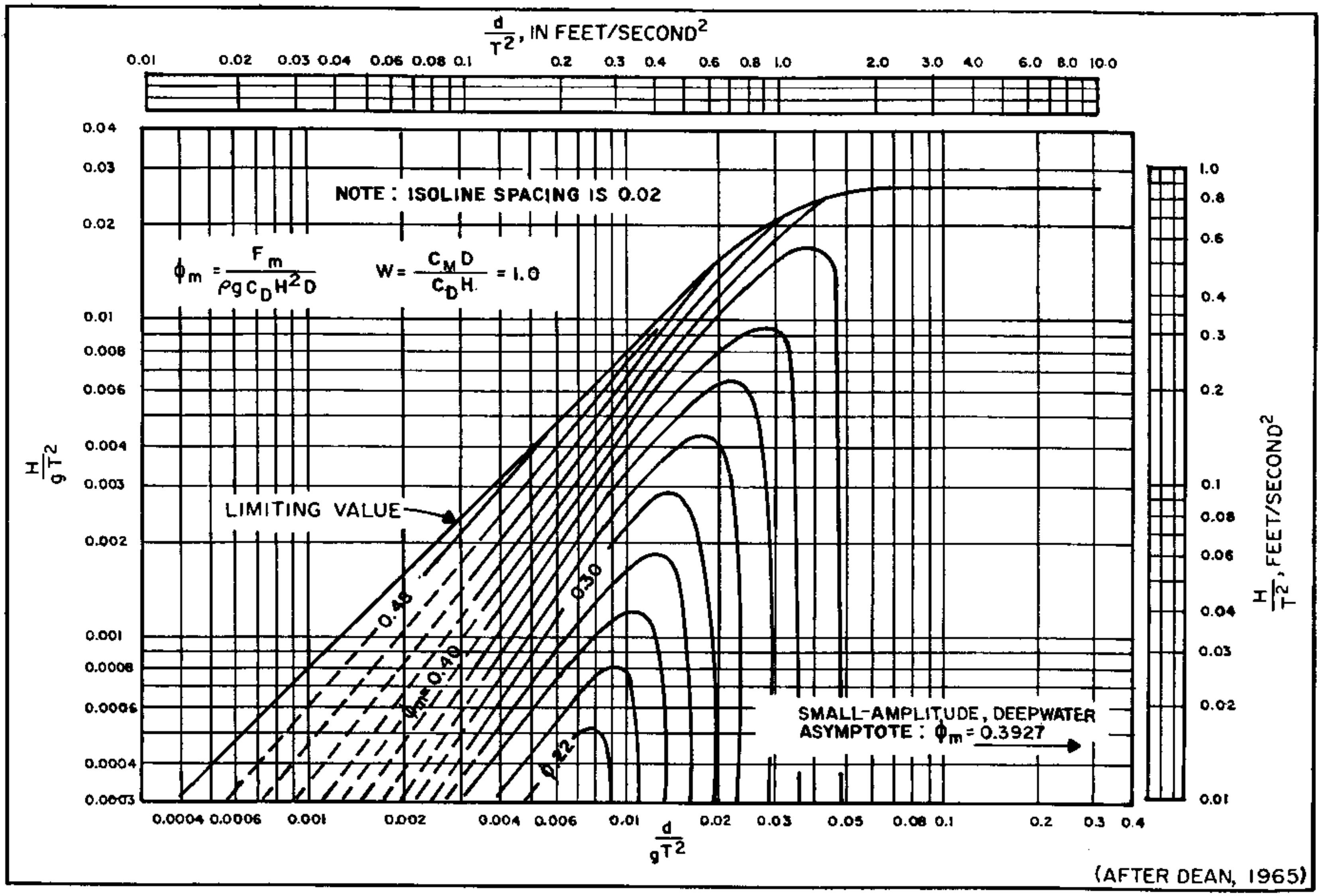


FIGURE 138
Isolines of ϕ_m as a Function of $d/g\,T^2$ and $H/g\,T^2$ for $W = 1.0$

TÀ2Ù for W = 1.0]

C_M = inertial, or added-mass, coefficient (obtained from Table 18)

c. Maximum Moment. The maximum moment about the mudline, M_{mDl}, is given by:

$$M_{mDl} = [\text{rho}]\, g\, C_D\, H^2\, D\, d\, [\text{alpha}]_m \qquad (7\text{-}14)$$

WHERE: [rho] = density of water

g = gravitational acceleration (32.2 feet per second^2)

C_D = drag coefficient (obtained from Table 17)

H = local wave height

D = pile diameter

d = water depth

$[\text{alpha}]_m$ = a coefficient given by Figures 139 through 142 as a function of $d/g T^2$ and $M/g T^2$ for various values of W

W is defined in Equation (7-13)

d. Reaction. The lever arm, z_{mDl}, is given by Equation (7-11), substituting z_{mDl} for z_{mD}, M_{mDl}, for M_{mD}, and F_{mDl} for F_{mD}.

EXAMPLE PROBLEM 39

Given:
- a. Equivalent unrefracted deepwater wave height, H'_o = 10 feet
- b. Water depth, d = 17 feet
- c. Wave period, T = 10 seconds
- d. Diameter of pile, D = 1 foot
- e. C_D = 0.7 and C_M = 2.0

Find: Maximum force and moment about mudline, considering both drag and inertial forces, and compare the results to those obtained in Example Problem 38; also find lever arm above the bottom.

Solution: (1) From Example Problem 37:

$$\frac{d}{g T^2} = 0.00528$$

$$\frac{H}{g T^2} = 0.00351$$

(2) Using Equation (7-13), W is:

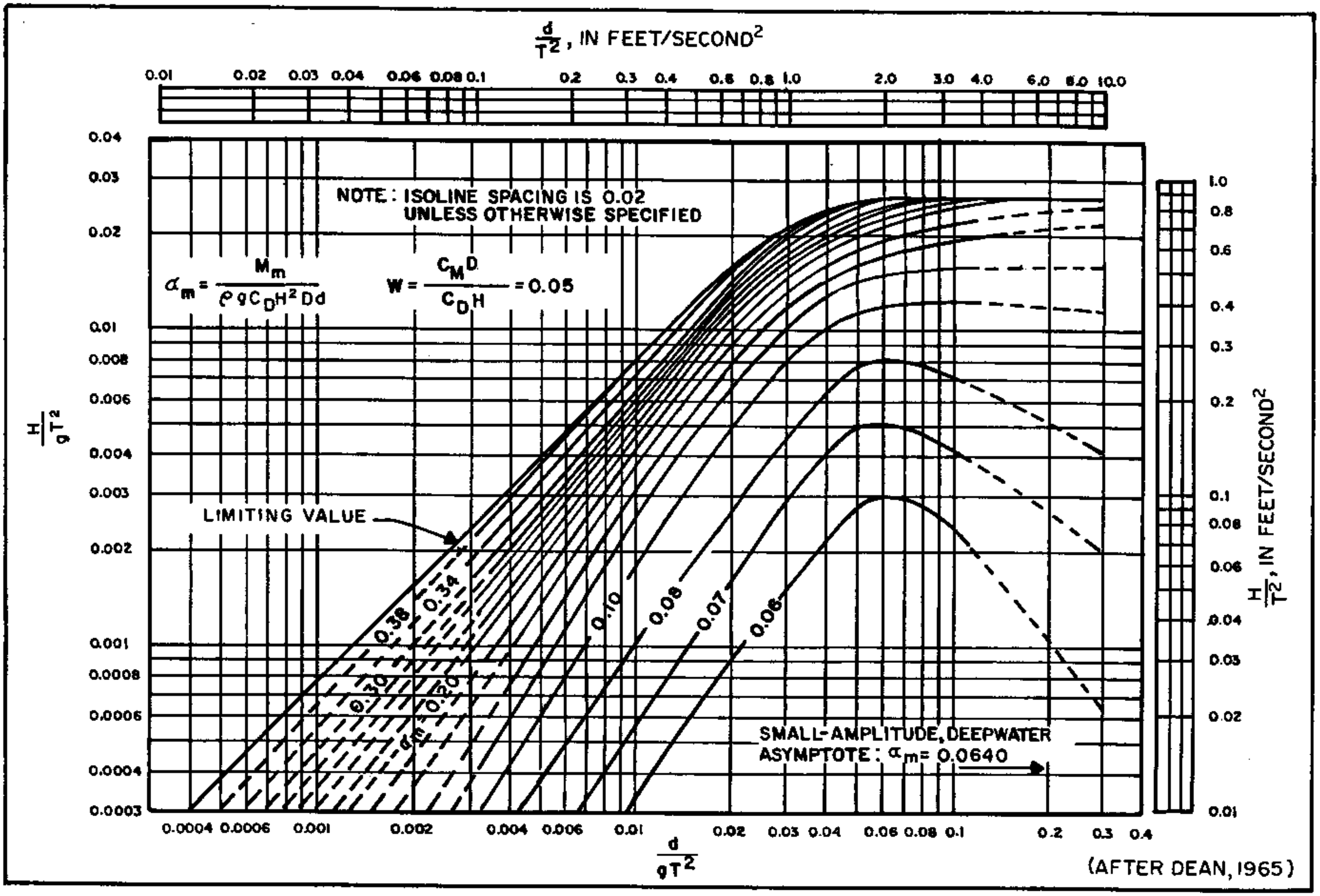


FIGURE 139
Isolines of α_m as a Function of $d/g\,T^2$ and $H/g\,T^2$ for $W = 0.05$

H/g TÀ2Ù for W = 0.05]

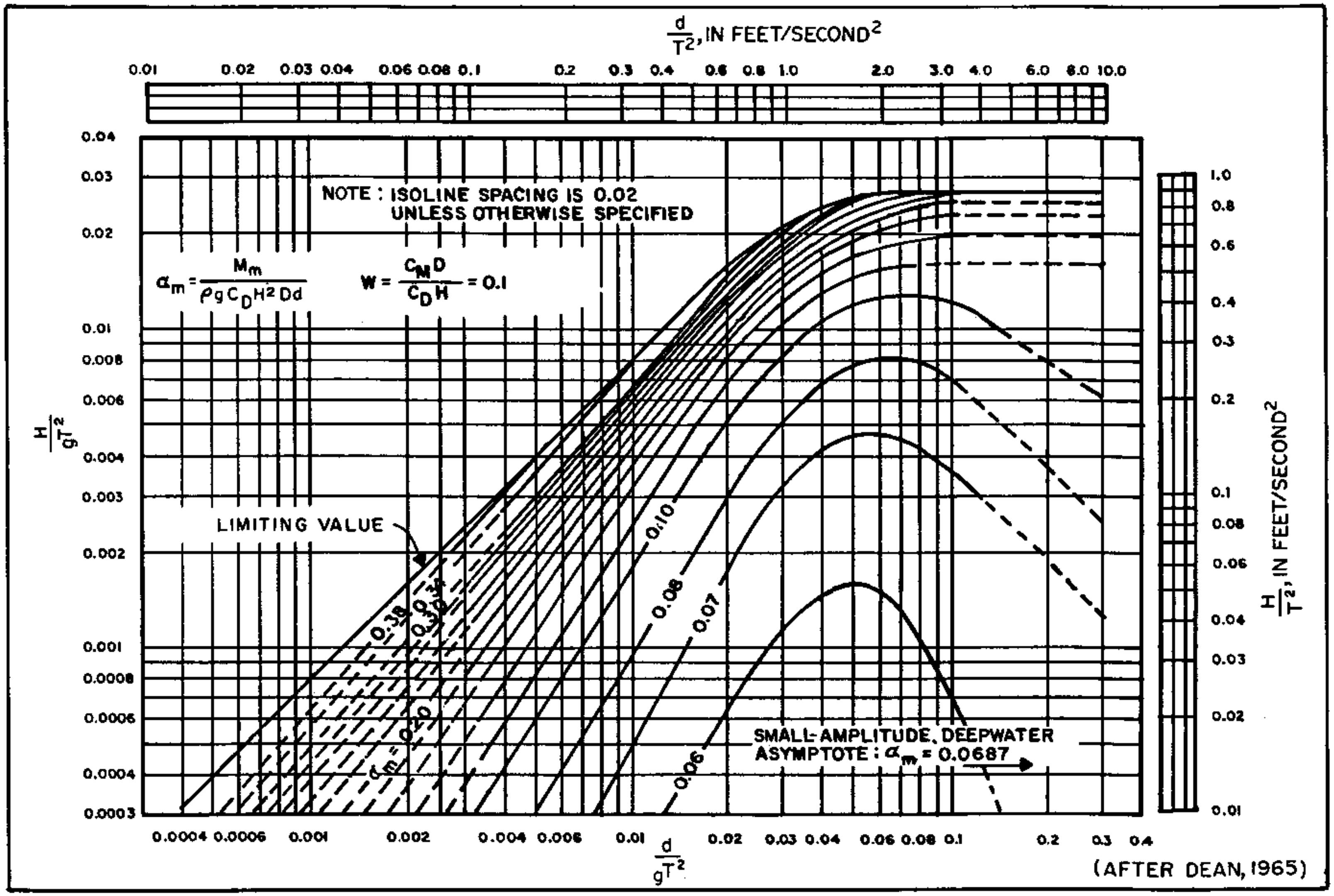


FIGURE 140
Isolines of α_m as a Function of d/gT^2 and H/gT^2 for W = 0.1

H/g TÀ2Ù for W = 0.1]

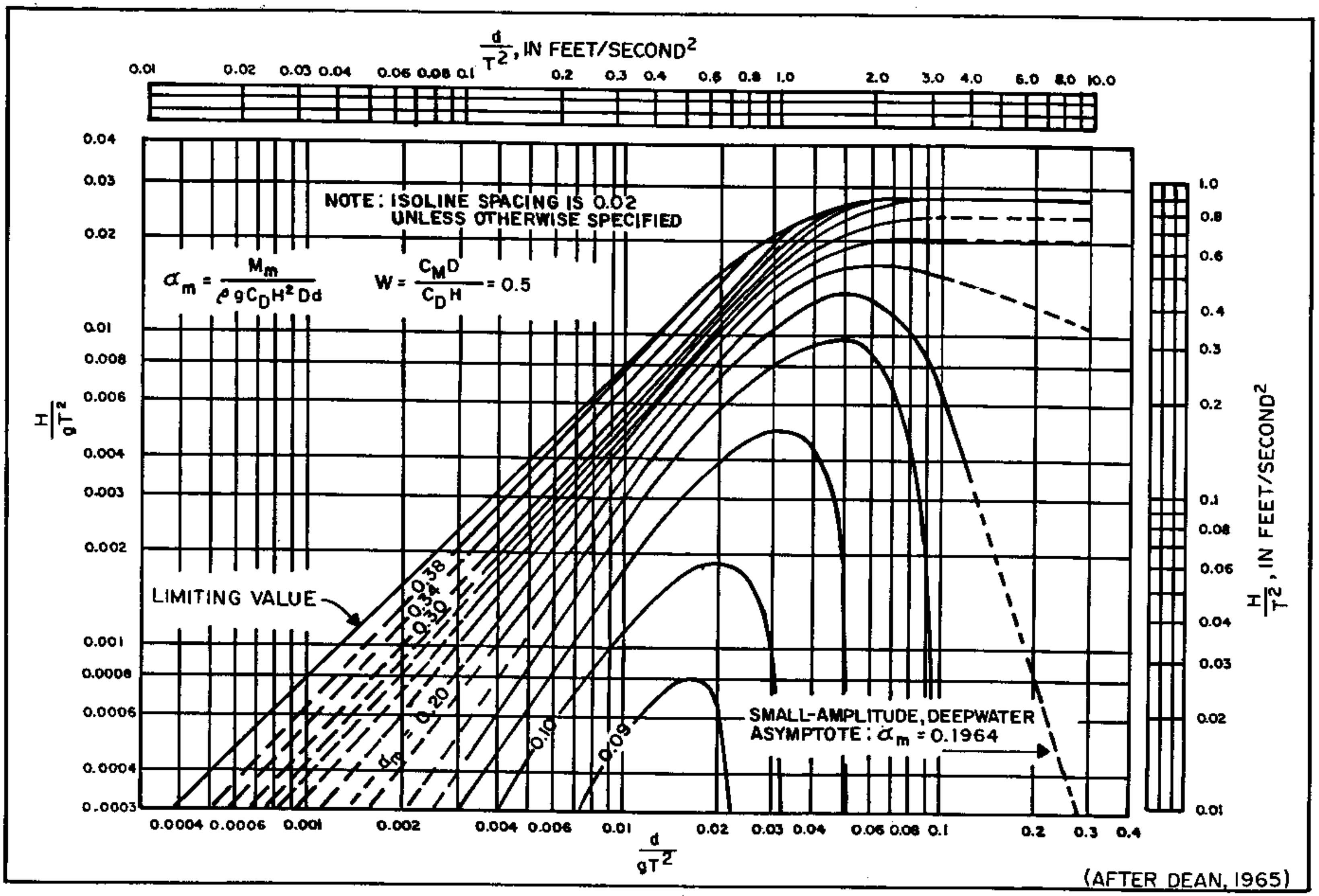


FIGURE 141
Isolines of α_m as a Function of d/gT^2 and H/gT^2 for W = 0.5

H/g TÀ2Ù for W = 0.5]

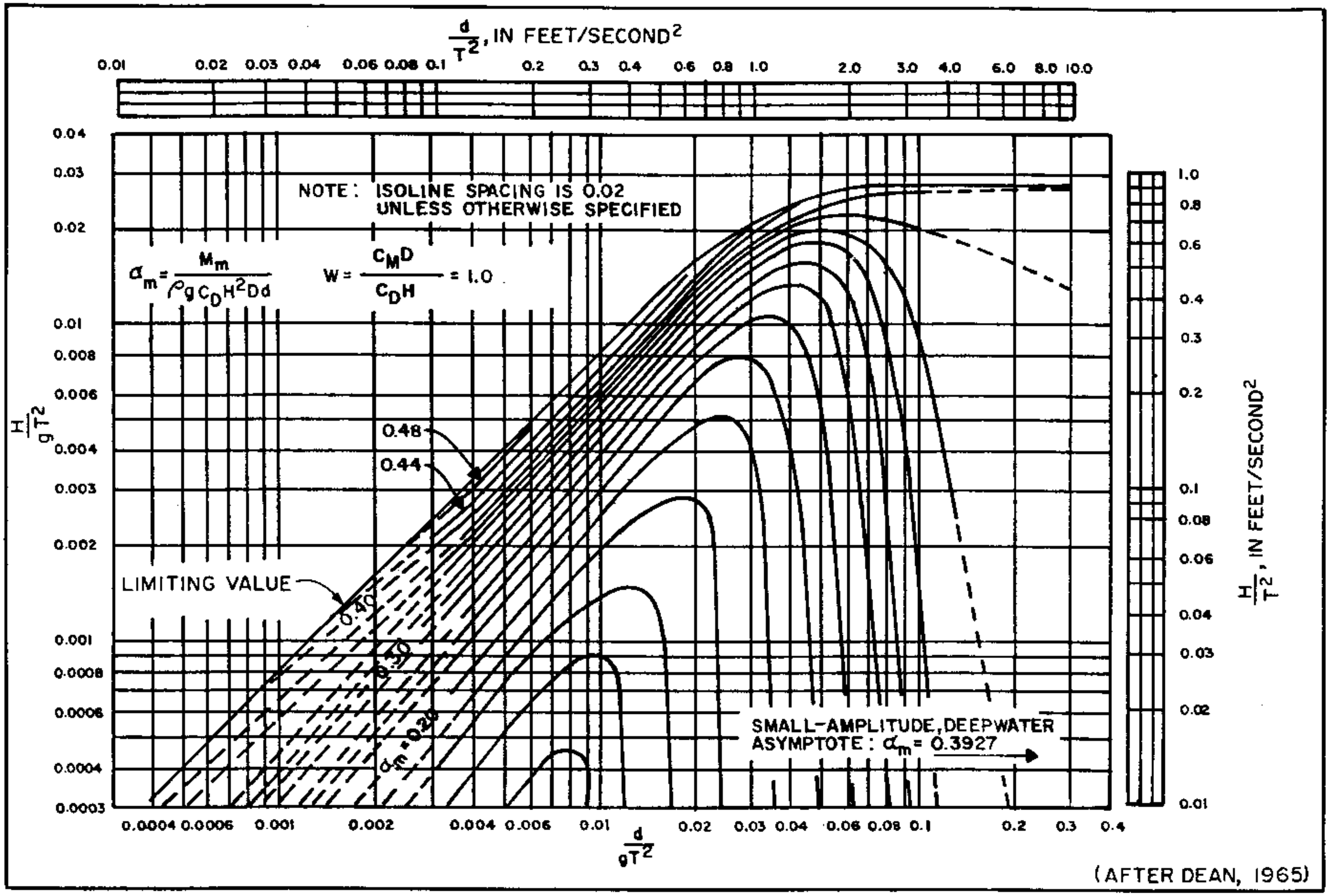


FIGURE 142

Isolines of α_m as a Function of $d/g\,T^2$ and $H/g\,T^2$ for W = 1.0

H/g TÀ2Ù for W = 1.0]

26.2-260

EXAMPLE PROBLEM 39 (Continued)

$$W = \frac{C_M D}{C_D H}$$

$$W = \frac{(2.0)\ (1)}{(0.7)\ (11.3)} = 0.253;\ \text{use } W = 0.25$$

(3) $[\phi]_m$ and $[\alpha]_m$ are found by interpolation of Figures 136 and 137, and 140 and 141, respectively, for $d/gT^2 = 0.00528$, $H/gT^2 = 0.00351$, and $W = 0.25$:

$[\phi]_m = 0.35$

$[\alpha]_m = 0.34$

(4) Using Equation (7-12), find F_{mDI}:

$F_{mDI} = [\rho]\ g\ C_D\ H^2\ D\ [\phi]_m$

(Assuming salt water, $[\rho]$ = 2 slugs per cubic foot.)

$F_{mDI} = (2)(32.2)(0.7)(11.3)^2(1)(0.35)$

F_{mDI} = 2,015 pounds

(5) Using Equation (7-14), find M_{mDI}:

$M_{mDI} = [\rho] g\ C_D\ H^2\ D\ d\ [\alpha]_m$

$M_{mDI} = (2)(32.2)(0.7)(11.3)^2\ (1)(17)(0.34)$

M_{mDI} = 33,271 foot-pounds

Note: The values obtained for force and moment are less than, hut compare reasonably well to (within the accuracy of the figures), those obtained in Example Problem 38. This indicates that the values in Case 1 provide conservative results and that effects of inertial forces are secondary.

(6) Using Equation (7-11), find z_{mDI} the lever arm above the bottom:

$$z_{mDI} = \frac{M_{mDI}}{F_{mDI}} = \frac{33{,}271}{2{,}015} = 16\text{-}5 \text{ feet}$$

7. CASE 3--MAXIMUM FORCE ON SINGLE PILE OF SMALL, NONUNIFORM DIAMETER (PRELIMINARY DESIGN). Piles of nonuniform diameter can be designed in special cases, or the nonuniformity may also occur naturally, as on a uniform diameter pile when marine fouling increases the diameter of the pile in the

intertidal zone. Marine fouling is primarily a function of biological activity. Nutrients, water temperature, salinity, turbidity, exposure to sunlight, wave activity, tides, and currents are factors which determine the fouling. Measurement of nearby existing pile structures should be made to determine the fouling characteristics, or, as a first approximation, the pile diameter should be increased by 1 foot within the range of low water to mean high tide. Nonuniformity of diameter also occurs when wooden piles are protected with a jacket about the waterline to prevent deterioration by marine borers. The method presented for Case 3 enables the designer to determine the effect of marine fouling or of a jacket on wave-induced forces.

Figure 143 is a definition sketch of a pile of three diameters. Element 2 represents marine fouling or a jacket in the intertidal zone. The method presented for Case 3 will give maximum forces and moments according to drag force only. In this case, the maximum force occurs under the crest. If inertial forces are important, then Case 4 and Case 5 must be used.

a. Range of Application. The range of application for Case 3 is the same as that for Case 1.

b. Force Calculations. The unit drag forces, f_D, are integrated from the bottom, $z = 0$, to the crest, $z = S$. The results of the integration are shown graphically in Figures 144, 145, and 146. The method used herein calculates the forces on each pile element.

c. Maximum Force. The maximum drag force, F_{mD} under the crest is given by:

$$F_{mD} = \left(\frac{1}{2}\right) \rho \, C_D \left(\frac{H}{T}\right)^2 d \, \phi_{DM} \left[(K_{DM})_{z=z_1} (D_1 - D_2) + (K_{DM})_{z=z_2} (D_2 - D_3) + (K_{DM})_{z=S_c} (D_3) \right] \quad (7\text{-}15)$$

WHERE: ρ = density of water

C_D = drag coefficient (obtained from Table 17)

H = local wave height

T = wave period

d = water depth

ϕ_{DM} = a nonlinear correction factor (determined from Figure 146)

$(K_{DM})_{z=z_i}$ is found in Figure 144 as a function of z/d and d/L_o

z_i = distance above bottom of point i

S_c = distance of free surface measured from the bottom to the wave crest when the crest is at the pile

D_i = respective pile diameter

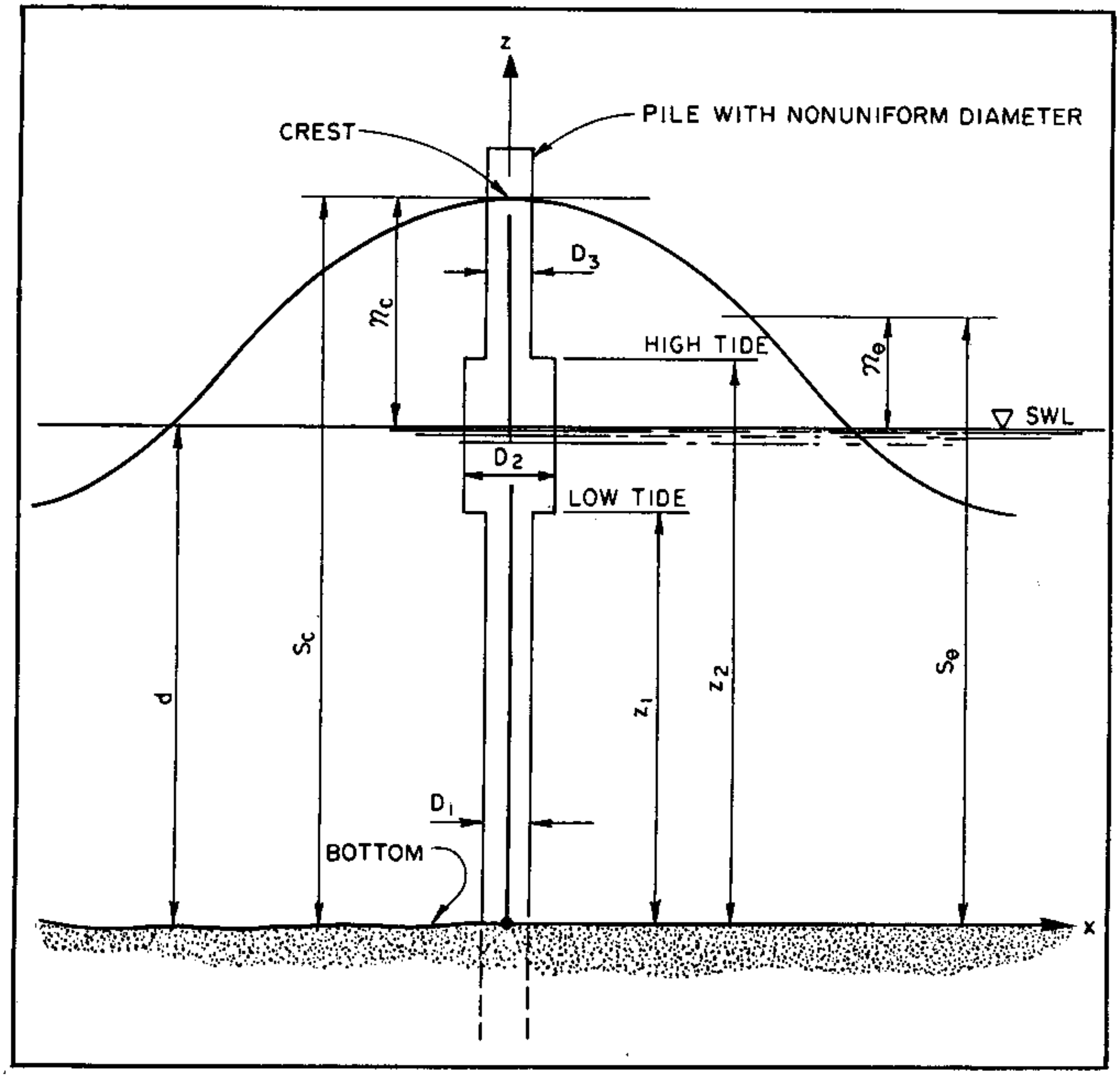


FIGURE 143
Definition of Terms for a Pile With Three Diameters

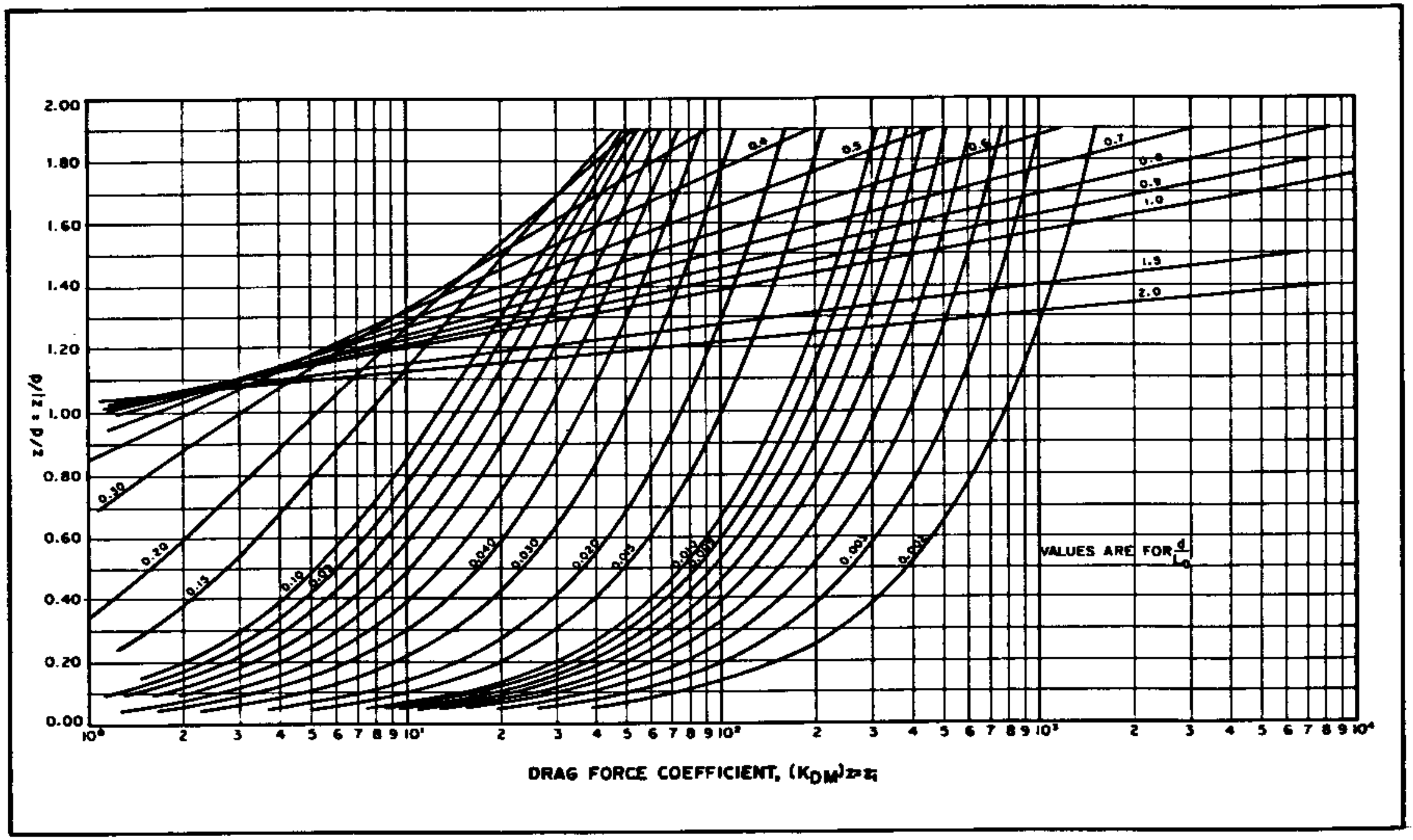


FIGURE 144

$(K_{DM})_{z=z_i}$ as a Function of z/d and d/L_o

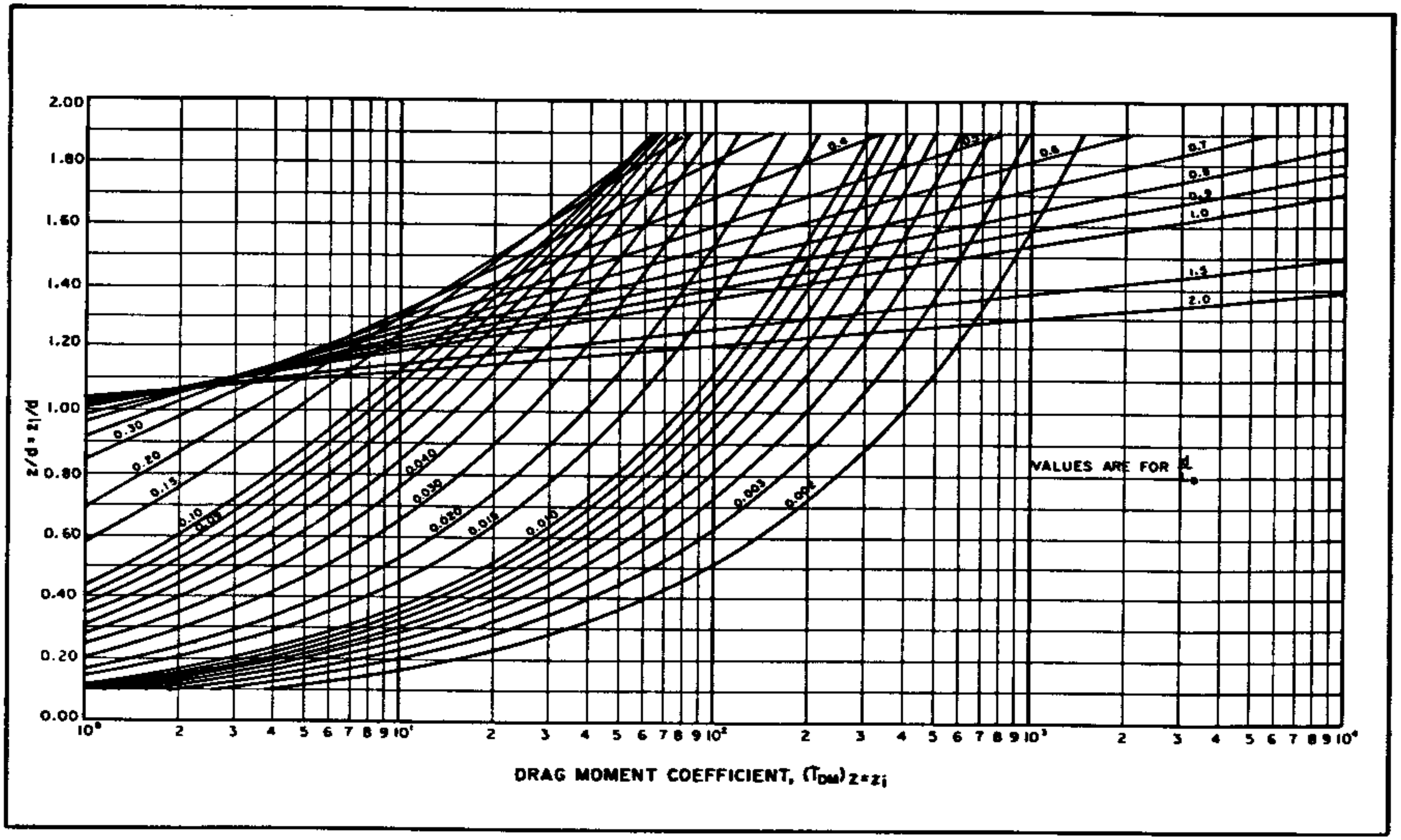


FIGURE 145

$(\tau_{DM})_{z=z_i}$ as a Function of z/d and d/L_o

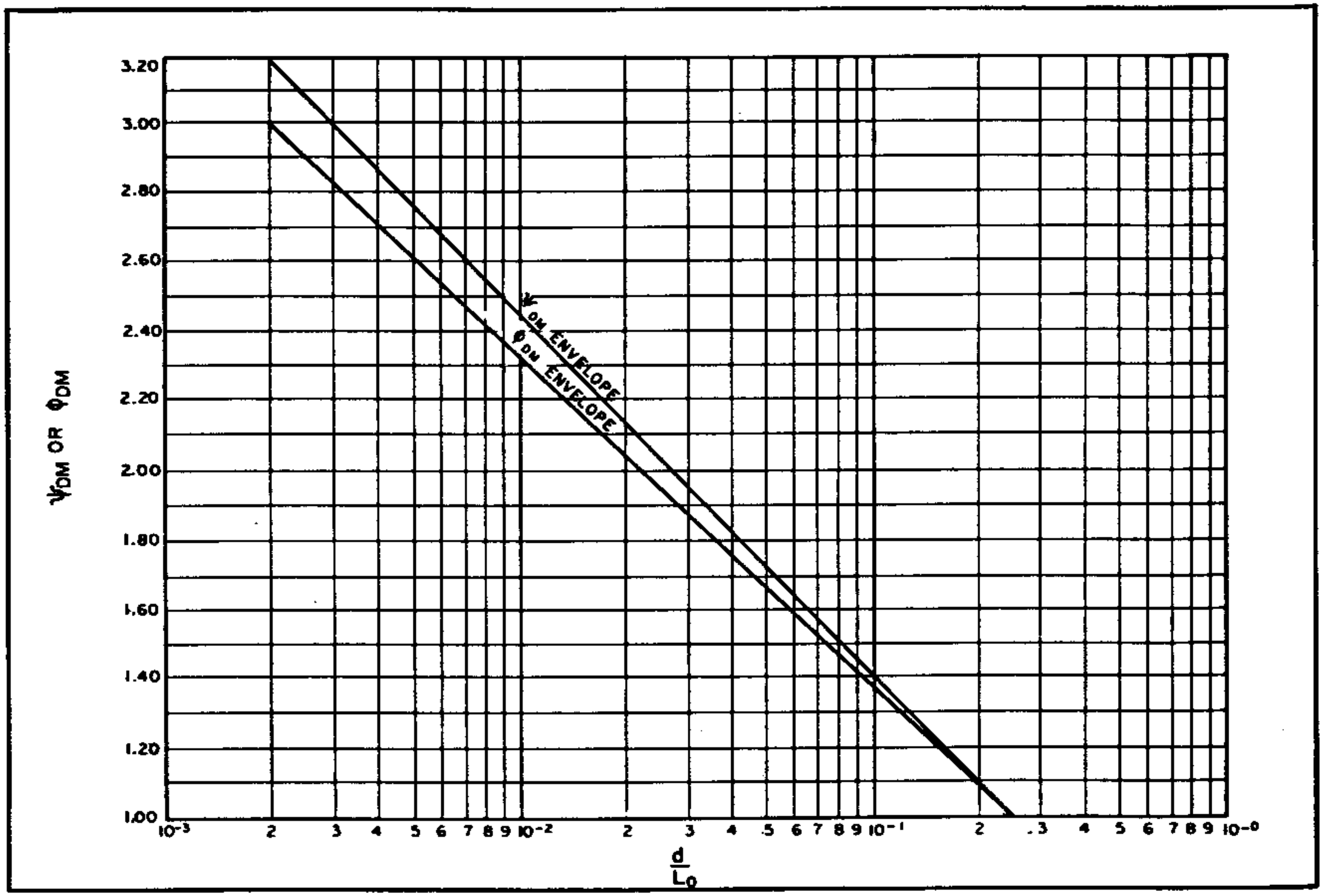


FIGURE 146
Envelopes of Nonlinear Correction Factors, Φ_{DM} and Ψ_{DM}, for Determining Maximum Drag Force and Moment, Respectively, as a Function of d/L_o

and [psi]ÚDM¿, for Determining Maximum Drag Force and Moment, Respectively, as a Function of d/LÚo¿]

d. Maximum Moment. The maximum drag moment, M_{mD}, can be determined by:

$$M_{mD} = \left(\frac{1}{2}\right) \rho \, C_D \left(\frac{H}{T}\right)^2 d^2 \, \psi_{DM} \left[(\tau_{DM})_{z=z_1} (D_1 - D_2) + (\tau_{DM})_{z=z_2} (D_2 - D_3) + (\tau_{DM})_{z=S_c} (D_3) \right] \qquad (7\text{-}16)$$

WHERE: ρ = density of water

C_D = drag coefficient (obtained from Table 17)

H = local wave height

T = wave period

d = water depth

ψ_{DM} = a nonlinear correction factor (determined from Figure 146)

$(\tau_{DM})_{z=z_i}$ is found in Figure 145 as a function of z/d and d/L_o

z_i = distance above bottom of point i

S_c = distance of free surface measured from the bottom to the wave crest when the crest is at the pile

e. Reaction. The lever arm, Z_{mD} is given by Equation (7-11).

EXAMPLE PROBLEM 40

Given:
a. Local wave height, H = 15 feet
b. Water depth, d = 20 feet
c. Wave period, T = 10 seconds
d. C_D = 0.7
e. Pile diameter is:

D_1 = 1.5 feet for z_o = 0 feet to z_1 = 18 feet
D_2 = 2 feet for z_1 = 18 feet to z_2 = 24 feet
D_3 = 1.5 feet for z_2 = 24 feet to z_3 = S_c

Find: The maximum drag force, maximum drag moment, and lever arm on the given pile of nonuniform diameter, and compare the results to those which would be obtained for a pile of uniform diameter, D = 1.5 feet.

Solution: (1) Calculate d/gT^2 and H/gT^2:

$$\frac{d}{g T^2} = \frac{20}{(32.2)(10)^2} = 0.00621$$

$$\frac{H}{g T^2} = \frac{15}{(32.2)(10)^2} = 0.00466$$

EXAMPLE PROBLEM 40 (Continued)

(2) Using Equation (7-8), find S_c:

(a) Find η_c:

From Figure 132 for $d/gT^2 = 0.00621$ and $\frac{H}{T^2} = 0.00466$:

$$\frac{\eta_c}{H} = 0.84$$

$\eta_c = (0.84)\ H\ (15) = 12.6$ feet

(b) $S_c\ \eta_c + d$

$S_c = 12.6 + 20 = 32.6$; use $S_c = 33$ feet

(3) Determine drag force and drag moment coefficients, (K_{DM}) and $((\tau_{DM})_{z=z_i})$, from Figures 144 and 145, respectively:

(a) Find d/L_o:

$L_o = (g/2\pi)\ T^2 = (32.2/2\pi)(10)^2 = 512$ feet

$$d/L_o = \frac{20}{512} = 0.0391;\ \text{use } d/L_o = 0.039$$

(b) The results obtained from Figures 144 and 145 for $d/L_o = 0.039$ and desired values of z_i/d are tabulated below:

z_i	z_i/d	$(K_{DM})_{z=z_i}$	$((\tau_{DM})_{z=z_i})$
$z_1 = 18$ ft	$18/20 = 0.90$	32	16
$z_2 = 24$ ft	$24/20 = 1.20$	47	30
$z_3 = S_c$	$33/20 = 1.65$	72	67

(4) Determine nonlinear correction factors, ϕ_{DM} and ψ_{DM} from Figure 146 for $d/L_o = 0.039$:

$\phi_{DM} = 1.76$

$\psi_{DM} = 1.83$

(5) Using Equation (7-15), find the maximum drag force for the nonuniform-diameter pile:

$$F_{mD} = \left(\frac{1}{2}\right) \rho\ C_D \left(\frac{H}{T}\right)^2 d\ \phi_{DM}$$

[(KÚDM¿)Úz=zÚ1¿¿ (DÚ1¿ - DÚ2¿) + (KÚDM¿)Úz=zÚ2¿¿
(DÚ2¿ - DÚ3¿) + (KÚDM¿)Úz=SÚc¿ (DÚ3¿)]

EXAMPLE PROBLEM 40 (Continued)

```
         1             15
FÚmD¿ = (Ä) (2) (0.7) (ÄÄ)À2Ù (20) (1.76) [ (32)
       2             10
(1. 5 - 2) + (47) (2 - 1.5) + (72) (1.5) ]
```

FÚmD¿ = 6,403 pounds

(6) Using Equation (7-16), find the maximum drag moment for the nonuniform-diameter pile:

```
         1              H
MÚmD¿ = (Ä) [rho] CÚD¿ (Ä)À2Ù dÀ2Ù [psi]ÚDM¿
         2              T
[ ([tau]ÚDM¿)Úz=zÚ1¿¿ (DÚ1¿ - DÚ2¿) +

([tau]ÚDM¿)Úz=zÚ2¿¿ (DÚ2¿ - DÚ3¿) +

( [tau]ÚDM¿)Úz=SÚc¿ (DÚ3¿) ]
```

```
          1              15
MÚmD¿ = ( Ä ) (2) (0.7) (ÄÄ)À2Ù (20)À2Ù (1.83)
          2              10
[(16) (1.5 - 2.0) + (30) (2.0 - 1.5) + (67) (1.5)]
```

MÚmD¿ = 123,937 foot-pounds

(7) Using Equation (7-11), find zÚmD¿:

```
        MÚmD¿
zÚmD¿ = ÄÄÄÄÄ
        FÚmD¿

        123,937
zÚmD¿ = ÄÄÄÄÄÄÄÄÄÄ
          6,403
```

zÚmD¿ = 19.4 feet

(8) Compare the above results to those for a uniform pile of diameter, D = 1.5 feet:

(a) In order to use Figures 144 and 145, substitute SÚc¿/d for z/d.

z/d = SÚc¿/d = 33/20 = 1.65

Then (KÚDM¿)Úz=SÚc¿¿ = 72

and ([tau]ÚDM¿)Úz=SÚc¿ = 67

(b) In order to determine the maximum drag force, Equation (7-15) is used, with zÚi¿ = SÚc¿:

```
         1              H
FÚmD¿ = (Ä) [rho] CÚD¿ (Ä)À2Ù d [phi]ÚDM¿
```

2 T

[(KÚDM¿)Úz=SÚc¿ (D)]

1 15
FÚmD¿ = (Ä) (2) (0.7) (ÄÄ)À2Ù (20) (1.76) [(72)
2 10
(1.5)]

FÚmD¿ = 5,988 pounds

EXAMPLE PROBLEM 40 (Continued)

(c) In order to determine the maximum drag moment, Equation (7-16) is used, with $z_i = S_c$:

$$M_{mD} = \frac{1}{2} \rho C_D \left(\frac{H}{T}\right)^2 d \, \psi_{DM}$$

$$\left[\left(\tau_{DM}\right)_{z=S_c} (D)\right]$$

$$M_{mD} = \frac{1}{2} (2) (0.7) \left(\frac{15}{10}\right)^2 (20)^2 (1.83)$$
$$[(67) (1.5)]$$

$$M_{mD} = 115{,}866 \text{ pounds}$$

THEREFORE: The values of maximum drag force and maximum drag moment are approximately 7 percent higher for the nonuniform-diameter pile than for the uniform-diameter pile.

(9) Using Equation (7-11), find z_{mD}:

$$z_{mD} = \frac{M_{mD}}{F_{mD}} = \frac{115{,}866}{5{,}988} = 19.3 \text{ feet}$$

8. CASE 4--WAVE FORCE, AT AN ARBITRARY WAVE-PHASE ANGLE, ON SINGLE PILE OF INTERMEDIATE, NONUNIFORM DIAMETER (PRELIMINARY DESIGN). Calculation of the wave force at an arbitrary wave-phase angle is required to estimate the maximum forces on both uniform- and nonuniform-diameter piles and to estimate forces on a combination of piles.

a. Range of Application. This case includes both drag and inertial forces and therefore makes no assumption regarding D/H or d/L_o. However, it does not apply to closely spaced piles ($\delta \leq 2D$), nor for D greater than 20 percent of the wavelength. This case deals with linear theory to illustrate the method. Case 5 incorporates nonlinear corrections for final design.

b. Linear Forces.

(1) Drag Force. The linear drag force, F_D, on a uniform-diameter pile as a function of phase angle is given by:

$$F_D = \frac{1}{2} \rho C_D D \left(\frac{H}{T}\right)^2 d \left(K_D\right)_z \qquad (7\text{-}17)$$

WHERE: ρ = density of water

C_D = drag coefficient (obtained from Table 17)

D = pile diameter

H = local wave height

T = wave period

d = water depth

(KÚD¿)Úz¿ = (KÚDM)z¿ cos [theta]^3 cos [theta]^3

θ = wave-phase angle

$(K_{DM})_z$ is found in Figure 144 as a function of z/d and d/L_o.

The linear drag force, F_D, on a nonuniform-diameter pile as a function of phase angle is given by:

$$F_D = \left(\frac{1}{2}\right) \rho C_D \left(\frac{H}{T}\right)^2 d \, (K_{DM})_{z=z_i} \quad (7\text{-}17)$$

$$+ (K_{DM})_{z=z_2} (D_2 - D_3) + (K_{DM})_{z=s_\theta} (D_3)] \; |\cos \theta| \cos \theta \quad (7\text{-}18)$$

WHERE: D_i = respective pile diameter

s_θ = distance of free surface measured from the bottom at an arbitrary wave-phase angle, θ

(2) Inertial Force. The linear inertial force, F_I, on a uniform-diameter pile as a function of phase angle is given by:

$$F_I = \rho C_M \pi \left|\frac{D^2}{4}\right| \left|\frac{H}{T^2}\right| d \, (K_I)_z \quad (7\text{-}19)$$

WHERE: C_M = inertial, or added-mass, coefficient (obtained from Table 18)

$(K_I)_z = (K_{IM})_z \sin \theta$

$(K_I)_z$ is found in Figure 147 as a function of z/d and d/L_o.

The linear inertial force, F_I, on a nonuniform-diameter pile as a function of phase angle is given by:

$$F_I = \rho C_M \left(\frac{H}{T^2}\right) d \, (K_{IM})_{z=z_i} (D_1^2 - D_2^2)$$

$$+ (K_{IM})_{z=z_2} (D_2^2 - D_3^2) + (K_{IM})_{z=S_\theta} (D_3^2)] \sin \theta \quad (7\text{-}20)$$

c. Linear Moments.

(1) Drag Moment. The linear drag moment, M_D, for a uniform-diameter pile as a function of phase angle is given by:

$$M_D = \left(\frac{1}{2}\right) \rho C_D D \left(\frac{H}{T}\right) d^2 (\tau_D)_z \quad (7\text{-}21)$$

WHERE: ρ = density of water
C_D = drag coefficient (obtained from Table 17)
D = pile diameter
H = local wave height
T = wave period

d = water depth

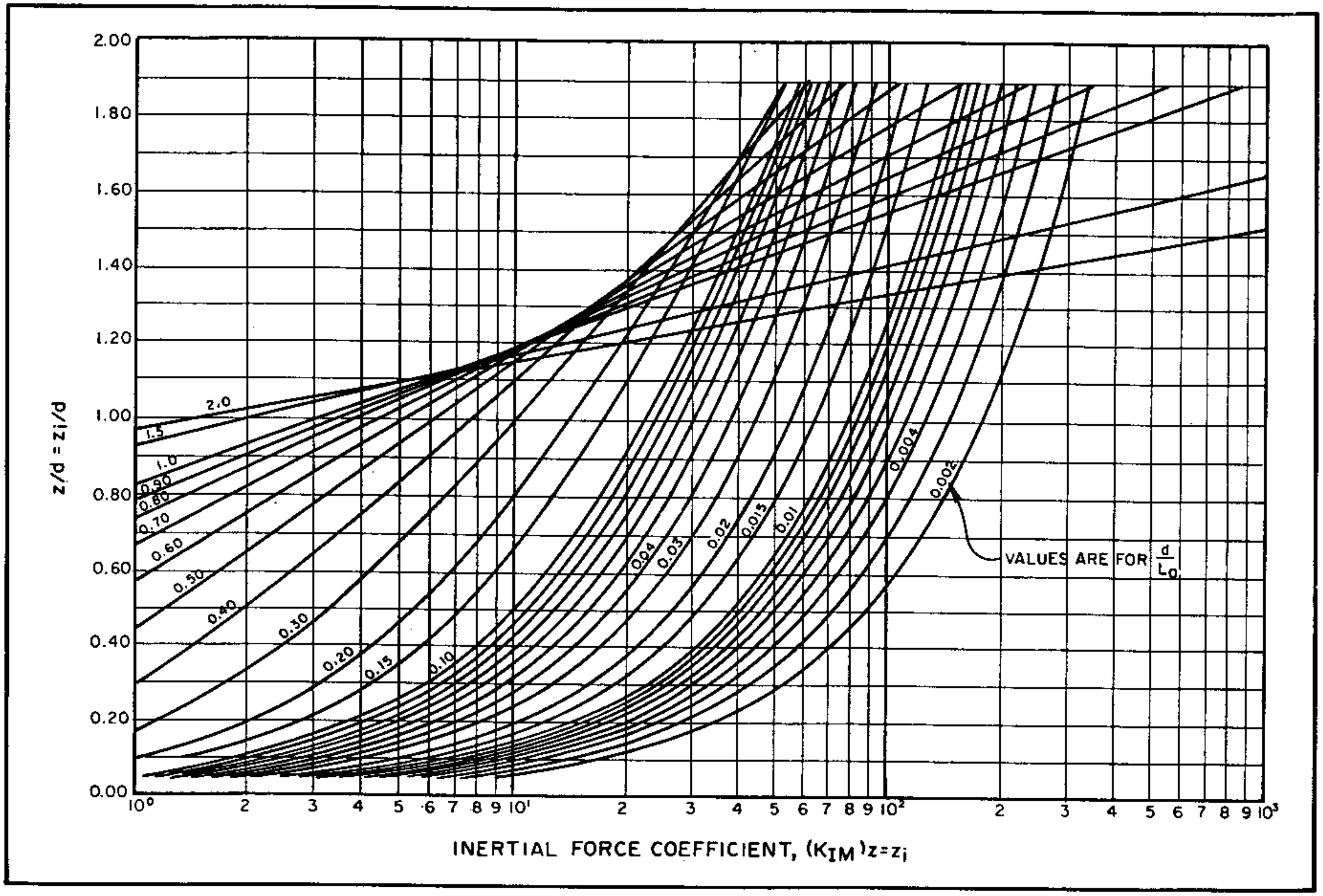


FIGURE 147

$(K_{IM})_z$ as a Function of z/d and d/L_o

([tau]ÚD)z¿ ([tauÚDM)z¿ cos [theta]³ cos [theta]³

([tau]ÚDM)z¿ is found in Figure 145 as a function of z/d and d/LÚo¿.

The linear drag moment, MÚD¿, on a nonuniform-diameter pile as a function of phase angle is given by:

```
            1             H
MÚD¿  = (Ä) [rho] CÚD¿ (Ä)À2Ù dÀ2Ù [(([tau]ÚDM¿)Úz=zÚ1¿¿ (DÚ1¿ - DÚ2¿)
            2             T
      + (([tau]ÚDM¿)Úz=zÚ2¿¿ (DÚ2¿ - DÚ3¿) + ([tau]ÚDM¿)Úz=sÚ[theta]¿

        (DÚ3¿)] cos [theta] ³ cos [theta]³                            (7-22)
```

WHERE: DÚi¿ = respective pile diameter

SÚ[theta]¿ = distance of free surface measured from the bottom at an arbitrary wave-phase angle,

[theta] = wave-phase angle

(2) Inertial Moment. The linear inertial moment, MÚI¿, for a uniform-diameter pile as a function of phase angle is given by:

```
                        [pi]          H
   MÚI¿ = [rho] CÚM¿ (ÄÄÄ-) DÀ2Ù ( ÄÄÄÄÄ ) dÀ2Ù ([tau]ÚI)z¿         (7-23)
                          4          TÀ2Ù
```

WHERE: CÚM¿ = inertial, or added-mass, coefficient (obtained from Table 18)

([tau]ÚI)z¿ = ([tau]ÚIM)z¿ sin [theta]

([tau]ÚIM)z¿ is found in Figure 148 as a function of z/d and d/LÚo¿.

The linear inertial moment, MÚI¿, on a nonuniform-diameter pile as a function of phase angle is given by:

```
                        [pi]       H
MÚI¿  = [rho] CÚM¿ ( ÄÄÄÄ ) (ÄÄÄÄÄÄ) dÀ2Ù [(([tau]ÚIM¿)Úz=zÚ1¿¿ (DÚ1¿À2Ù -
                          4      TÀ2Ù

DÚ2¿À2Ù) + ([tau]ÚIM¿)Úz=zÚ2¿¿ (DÚ2¿À2Ù - DÚ3¿À2Ù) +
([tau]ÚDM¿)Úz=sÚ[theta]¿ (DÚ3¿À2Ù)] sin [theta] ³                      (7-22)
```

d. Phases. Figure 149 plots an example of the phase relationships among the drag, inertial, and total forces. The drag components are functions of cos [theta]³ cos [theta]³; therefore, they are positive between -90 deg. < [theta] 90 deg. and are negative otherwise. Drag components are positive when the free surface is above the still water level and they are symmetrical about the wave crest. The drag components are maximum under the wave crest. The inertial components are a function of sin [theta]; therefore they are positive in front of the wave crest for [theta] values between 0 and 180, and negative behind the wave crest. The total forces and total moments are obtained by adding drag and inertial components in front of the wave crest for [theta] values between 0 deg. and 90 deg. Behind the wave crest, the inertial components are subtracted from the drag components.

(1) Total Force. The value of [theta] for which the total force is maximum is, according to linear theory:

$$\theta = \theta_{Fm} = \sin^{-1} \left\{ \left(\frac{\pi}{4}\right)\left(\frac{D}{H}\right)\left(\frac{C_M}{C_D}\right) \left[\frac{(K_{IM})_{z=S_c}}{(K_{DM})_{z=S_c}} \right] \right\} \qquad (7\text{-}25)$$

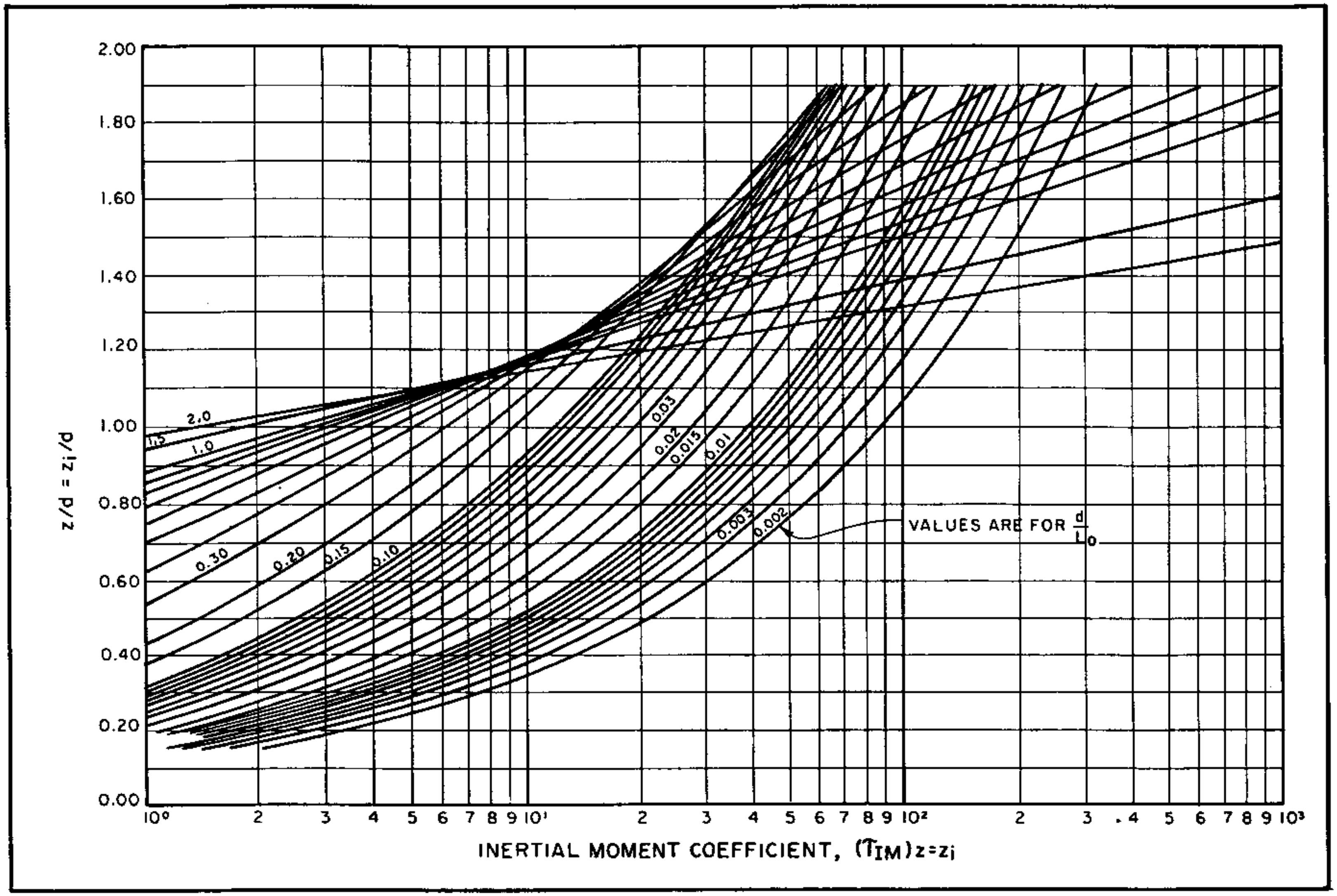


FIGURE 148
$(\tau_{IM})_z$ as a Function of z/d and d/L_o

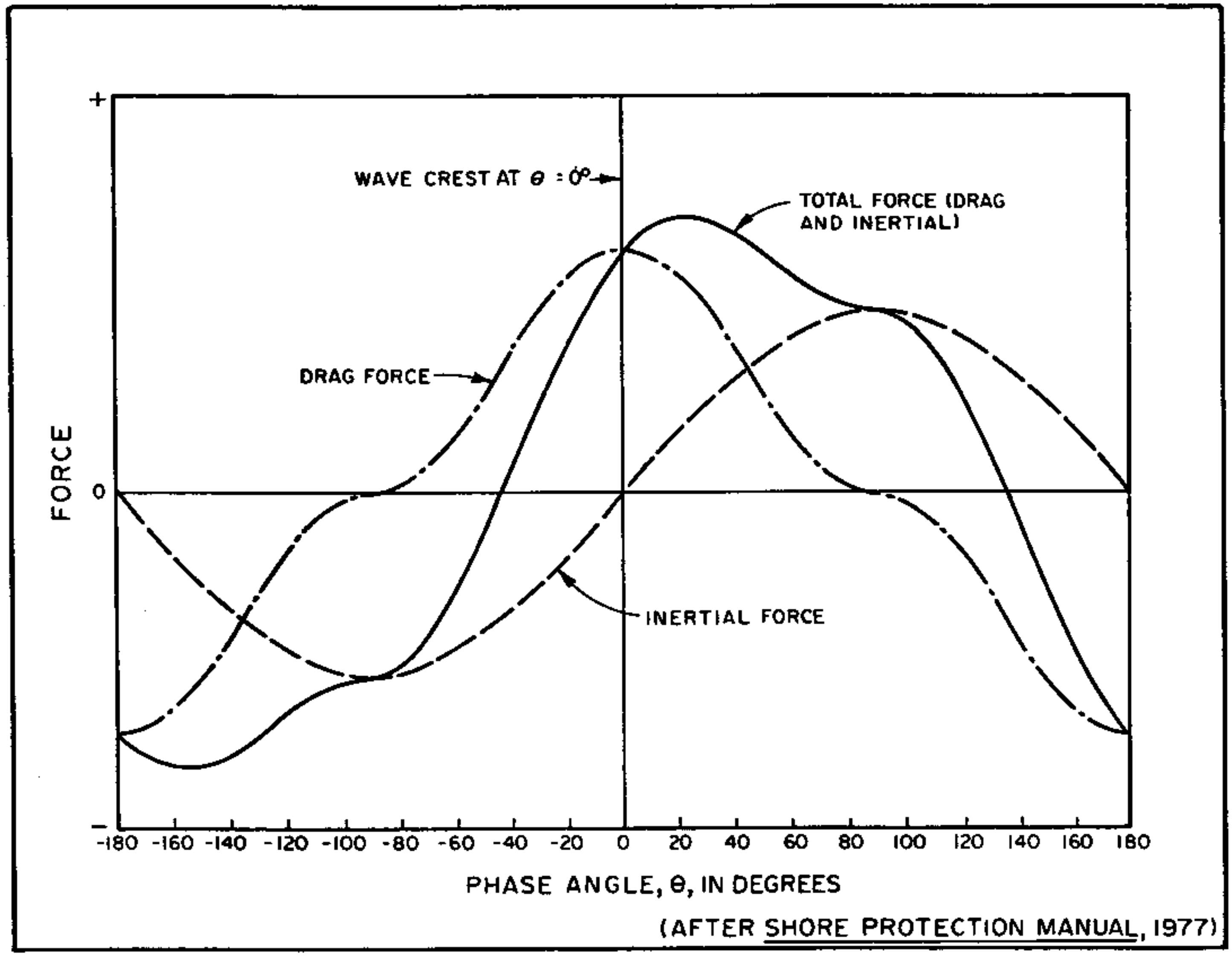


FIGURE 149
Phase Relationships Among Drag, Inertial, and Total Forces

Forces]

WHERE: θ = wave-phase angle

θ_{Fm} = wave-phase angle for which total Force is maximum

D = pile diameter

H = local wave height

C_M = inertial, or added-mass, coefficient (obtained from Table 18)

C_D = drag coefficient (obtained from Table 17)

$(K_{IM})_{z=S_c}$ is found in Figure 147 as a function of z/d and d/L_o.

$(K_{DM})_{z=S_c}$ is found in Figure 144 as a function of z/d and d/L_o.

S_c = distance of free surface measured from the bottom to the wave crest when the crest is at the pile

(2) Total Moment. The total moment is maximum when:

$$\theta = \theta_{Mm} = \sin^{-1}\left\{\left(\frac{\pi}{4}\right)\left(\frac{D}{H}\right)\frac{C_M}{C_D}\left[(\tau_{IM})_{z=S_c}\right]\right\} \qquad (7\text{-}26)$$

WHERE: θ_{Mm} = wave-phase angle for which total moment is maximum

$(\tau_{IM})_{z=S_c}$ is found in Figure 148 as a function of z/d and d/L_o.

$(\tau_{DM})_{z=S_c}$ is found in Figure 145 as a function of z/d and d/L_o.

The values of θ for maximum force and moment are close to one another, but are not identical. However, both occur before the wave crests.

e. Reaction. The lever arm, z_{mDI}, is given by Equation (7-11), substituting z_{mDI} for z_{mD}, M_{mDI} for M_{mD} and F_{mDI} for F_{mD} for $\theta = \theta_{Fm}$.

f. Example of Application. An example of the application of Case 4 is provided at the end of Subsection 7.9, CASE 5--NONLINEAR CORRECTIONS, with the nonlinear corrections shown.

9. CASE 5--NONLINEAR CORRECTIONS FOR MAXIMUM WAVE FORCES ON SINGLE PILES OF NONUNIFORM DIAMETER (FINAL DESIGN).

a. Definitions. The application of the linear theory gives a good order of magnitude of forces and moments acting on a pile or a combination of piles. However, for more accuracy and safety, nonlinear corrections may be important. These corrections apply to:

(1) the definition of the free surface;
(2) the definition of the velocity and acceleration field; and
(3) the variation of the submerged mass of the pile.

Item (3), which is of secondary importance, occurs near the breaking zone. The correction is necessary because of a rapid change in the submerged mass of the pile, due to the rapid change in free surface. This correction is usually small and, for this reason, a procedure for correcting for the variation in submerged mass is not included in this manual. The case of breaking waves is treated in Subsection 7.12, CASE 8--FORCES DUE TO BREAKING WAVES.

b. Corrections Due to Nonlinear Free Surface. The first nonlinear correction is due to the fact that the free surface is not defined by a sinusoid, as it is in linear theory, but is rather defined by a complex function of the phase angle, [theta], and the relative wave height, H/HÚb¿. HÚb¿ is the limiting height of a wave over the bottom. The limiting wave height is H = 0.78 d for a flat bottom; HÚb¿ can be determined from Figure 42 for a sloped bottom. The relative free surface, SÚ[theta]¿/d, is given as a function of H/HÚb¿, d/LÚo¿, and [theta] in Figure 150.

EXAMPLE PROBLEM 41

Given:
a. Equivalent unrefracted deepwater wave height, H'Úo¿ = 10 feet
b. Water depth, d = 17 feet
c. Wave period, T = 10 seconds
d. Bottom slope m = 0.02

Find: Nonlinear free surface at phase angle, [theta] = 20 deg.

Solution: (1) From Example Problem 36:

d/LÚo¿ = 0.0332

H = 11.3 feet, HÚb¿ = 12 feet, and H/HÚb¿ = 0.94

(2) Using Figure 150, enter the d/LÚo¿ abscissa with d/LÚo¿ = 0.0332; at the point of intersection with the H/HÚb¿ curves encompassing the given value of H/HÚb¿ = 0.94 in this case, H/HÚb¿ = 1.00 and H/HÚb¿ = 0.75.), lines are drawn horizontally to the right until intersection with the corresponding [theta] curves. (Note that each H/HÚb¿ curve has its own set of [theta] curves.) Finally, vertical lines are drawn from the intersection points on the [theta] curves to give two values for SÚ[theta]¿/d from which the correct value of SÚ[theta]¿/d is found by interpolation:

(a) When [theta] = 20 deg. and H/HÚb¿ = 1.00:

SÚ[theta]¿/d 1.266

(b) When [theta] = 20 deg. and H/HÚb¿ = 0.75:

SÚ[theta]¿/d = 1.256

(c) By interpolation:

When [theta] = 20 deg. and H/HÚb¿ = 0.94:

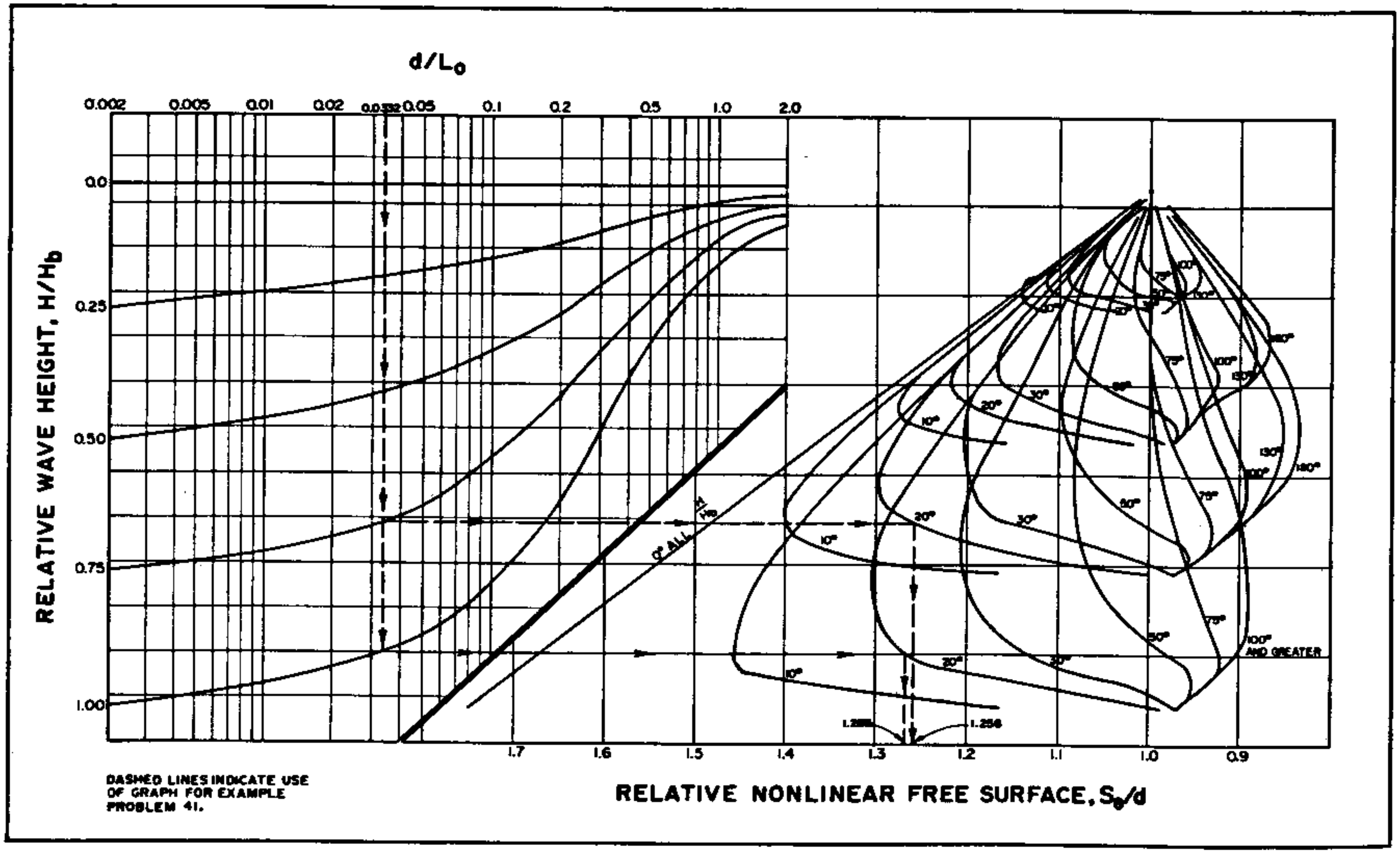


FIGURE 150
Relative Nonlinear Free Surface, S_θ/d, as a Function of H/H_b, d/L_o, and θ

Function of H/HÚb¿, d/LÚo¿, and [theta]

26.2-278

EXAMPLE PROBLEM 41 (Continued)

$S_\theta/d = 1.264$; use $S_\theta/d = 1.26$

(3) Find S_θ:

$S_\theta/d = 1.26$

$S_\theta = 1.26\ d$

$S_\theta = (1.26)(17) = 21.4$ feet

THEREFORE: The free-surface elevation is 21.4 feet above the bottom, or $S_\theta - \eta_\theta = 21.4 - 17 = 4.4$ feet above the still water level.

c. Correction of Forces and Moments Due to Nonlinear Velocity and Acceleration Fields. Nonlinear correction factors are applied to the forces and moments determined by linear theory in Subsection 7.8 (Case 4) using the nonlinear distance of the free surface above the bottom determined in Subsection 7.9.b., Corrections Due to Nonlinear Free Surface, above. Conservative estimates of the nonlinear correction are made by choosing its value at the free surface and neglecting any correction less than unity. Then the nonlinear forces and moments as a function of phase angle are determined as follows:

$$F_{D_{S_\theta}} = F_D\ \phi_D \qquad (7\text{-}27)$$

$$F_{I_{S_\theta}} = F_I\ \phi_I \qquad (7\text{-}28)$$

$$M_{D_{S_\theta}} = M_D\ \psi_D \qquad (7\text{-}29)$$

$$M_{I_{S_\theta}} = M_I\ \psi_I \qquad (7\text{-}29)$$

WHERE: S_θ subscript refers to values at an arbitrary phase angle, θ.

$F_{D_{S_\theta}}$ = drag force corrected for nonlinear effects

ϕ_D = drag-force correction factor for nonlinear velocity and acceleration fields (found in Figures 151-154 as a function of d/L_o, H/H_b, and θ)

$F_{I_{S_\theta}}$ = inertial force corrected for nonlinear effects

ϕ_I = inertial-force correction factor for nonlinear velocity and acceleration fields (found in Figures 155-158 as a function of d/L_o, H/H_b, and θ)

$M_{D_{S_\theta}}$ = drag moment corrected for nonlinear effects

ψ_D = drag-moment correction factor for nonlinear velocity and acceleration fields (found in Figures 159-162 as a function of d/L_o, H/H_b, and θ)

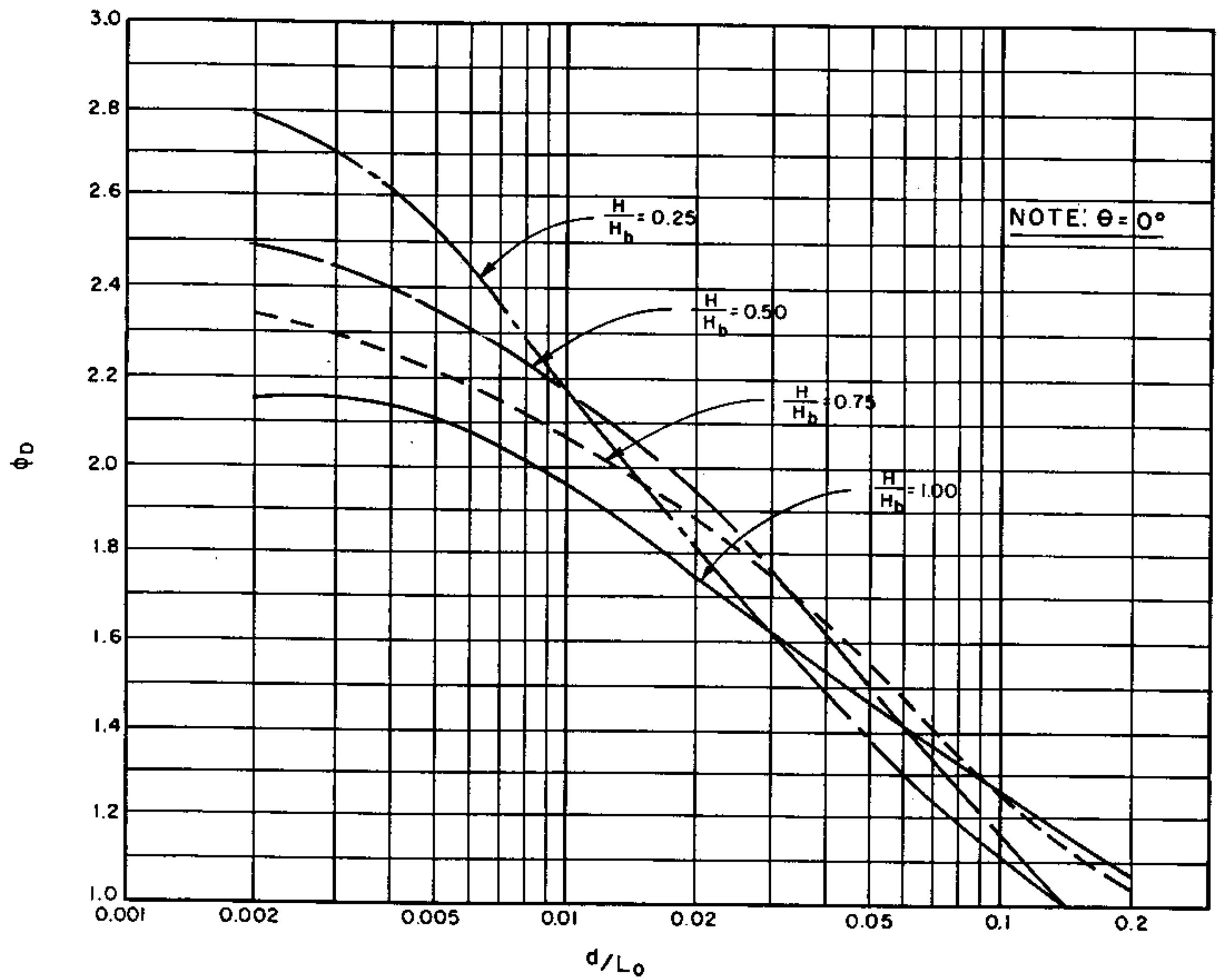


FIGURE 151
Nonlinear Drag-Force Correction Factor, ϕ_D, as a Function of d/L_o and H/H_b for $\theta = 0°$

Function of d/LÚo¿ and H/HÚb¿ for [theta] = 0 deg.]

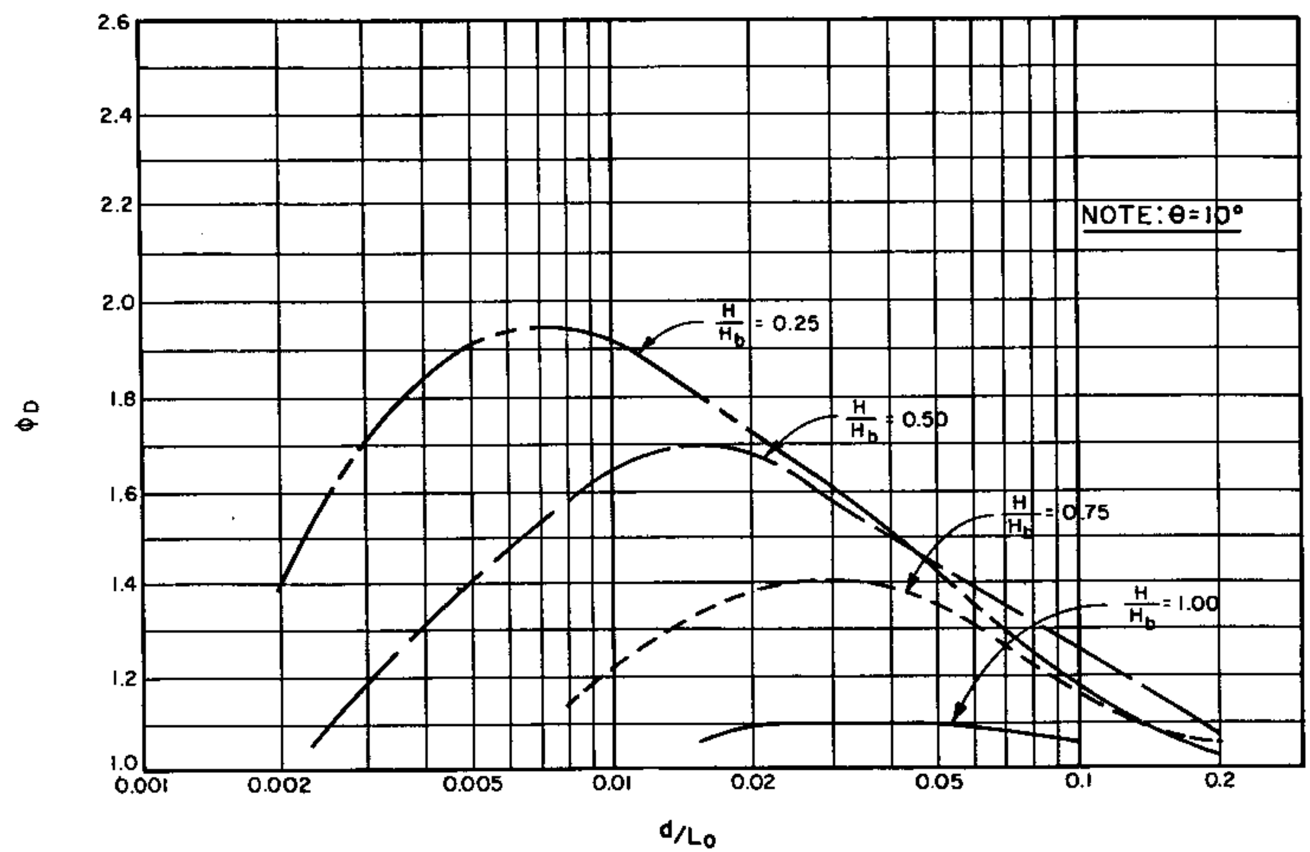


FIGURE 152
Nonlinear Drag-Force Correction Factor, ϕ_D, as a Function of d/L_o and H/H_b for $\theta = 10°$

Function of d/LÚo¿ and H/HÚb¿ for [theta] = 10 deg.]

26.2-281

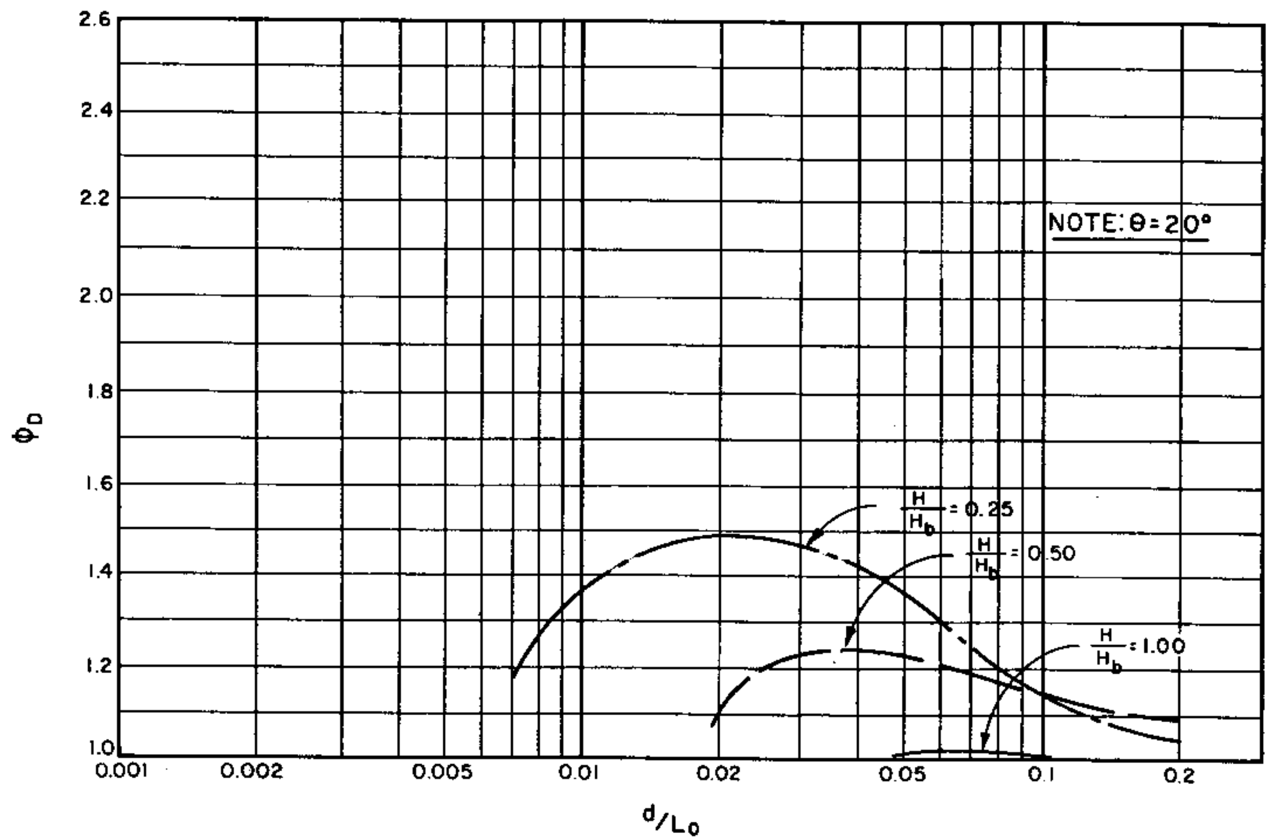


FIGURE 153
Nonlinear Drag-Force Correction Factor, ϕ_D, as a Function of d/L_o and H/H_b for $\theta = 20°$

Function of d/LÚo¿ and H/HÚb¿ for [theta] = 20 deg.]

26.2-282

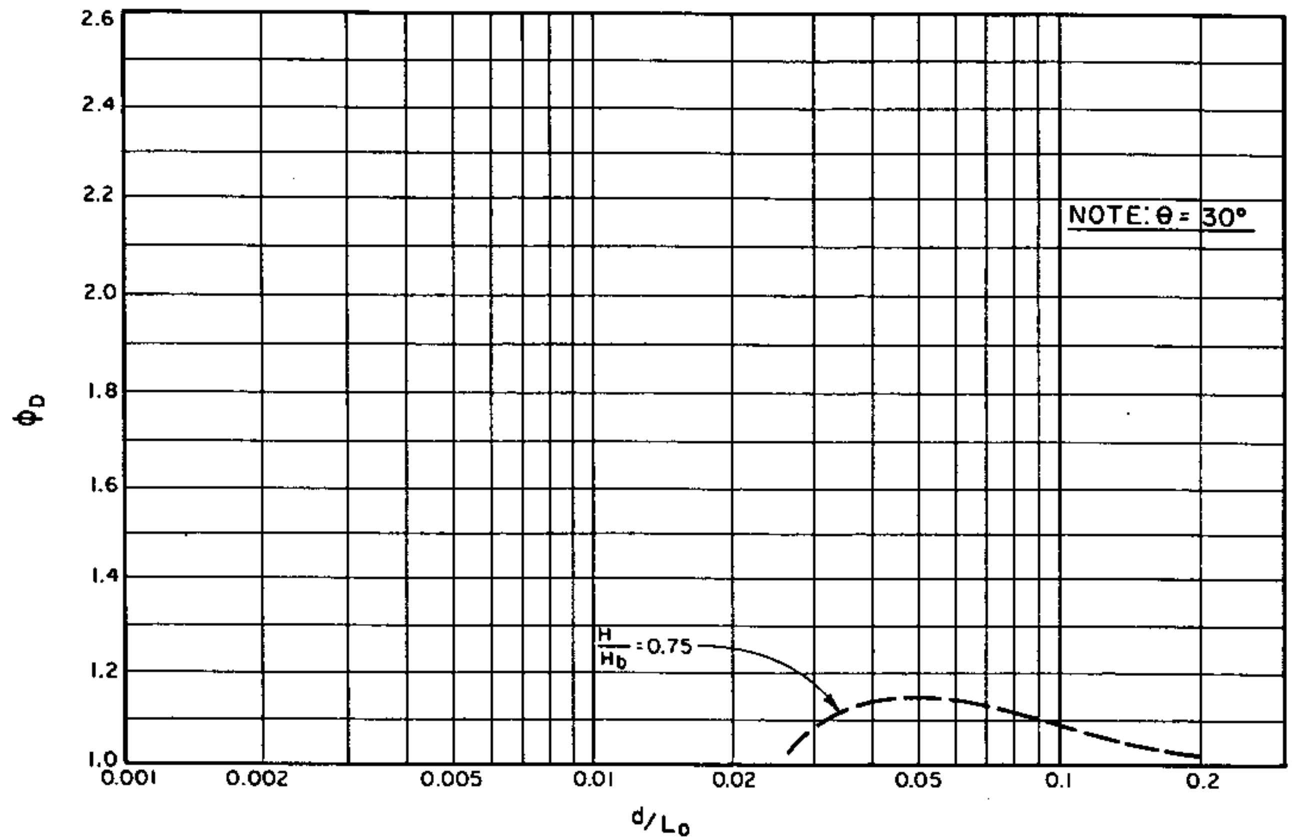


FIGURE 154
Nonlinear Drag-Force Correction Factor, ϕ_D, as a Function of d/L_o and H/H_b for $\theta = 30°$

Function of d/LÚo¿ and H/HÚb¿ for [theta] = 0 deg.]

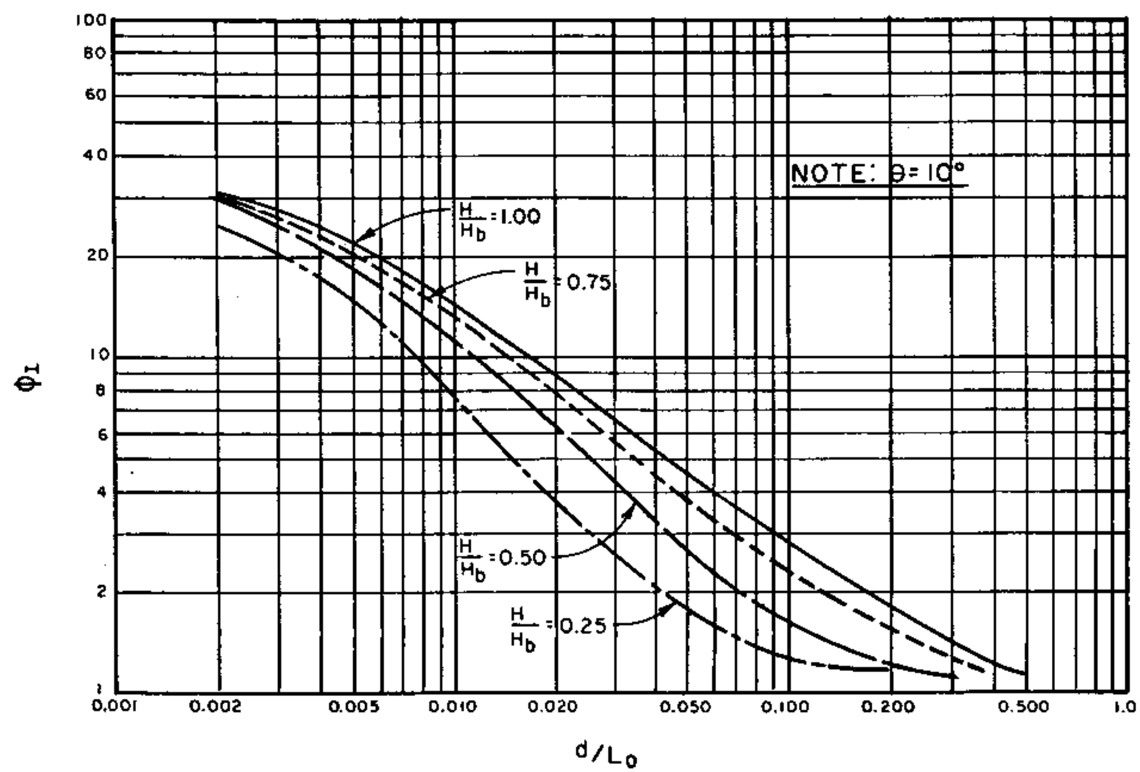


FIGURE 155
Nonlinear Inertial-Force Correction Factor, ϕ_I, as a Function of d/L_o and H/H_b for $\theta = 10°$

Function of d/LÚo¿ and H/HÚb¿ for [theta] = 10 deg.]

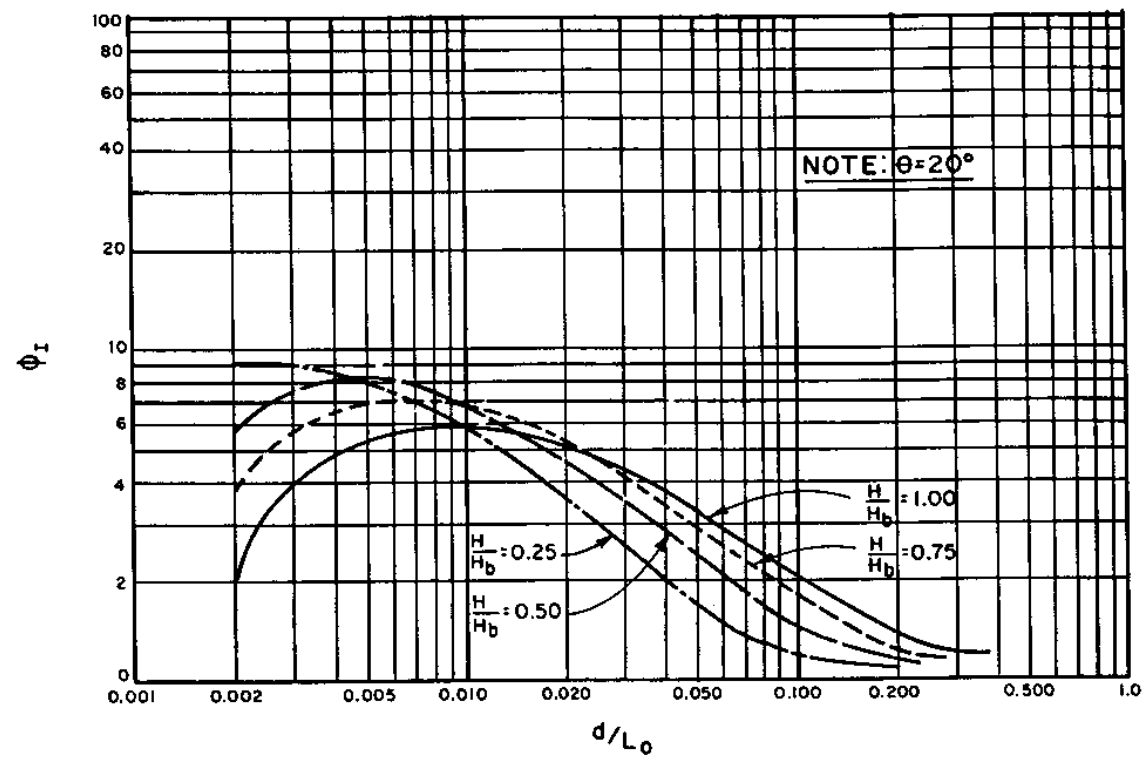


FIGURE 156

Nonlinear Inertial-Force Correction Factor, ϕ_I, as a Function of d/L_o and H/H_b for $\theta = 20°$

Function of d/LÚo¿ and H/HÚb¿ for [theta] = 20 deg.]

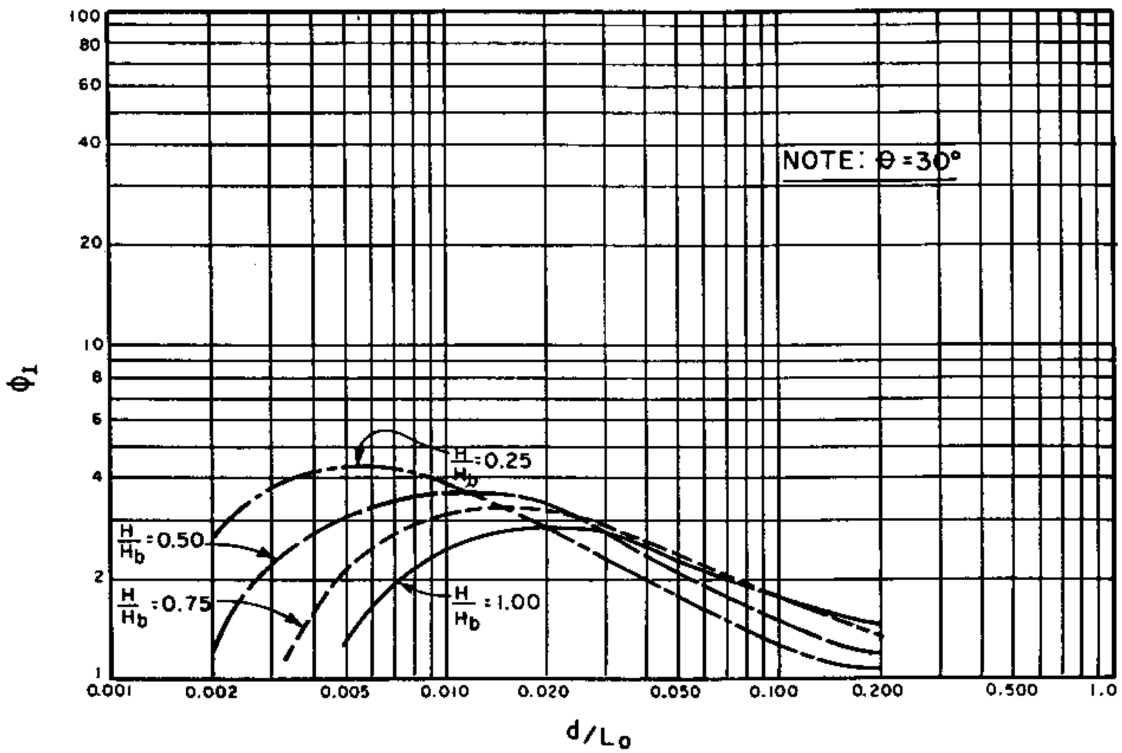


FIGURE 157
Nonlinear Inertial-Force Correction Factor, ϕ_I, as a Function of d/L_o and H/H_b for $\theta = 30°$

Function of d/LÚo¿ and H/HÚb¿ for [theta] = 30 deg.]

26.2-286

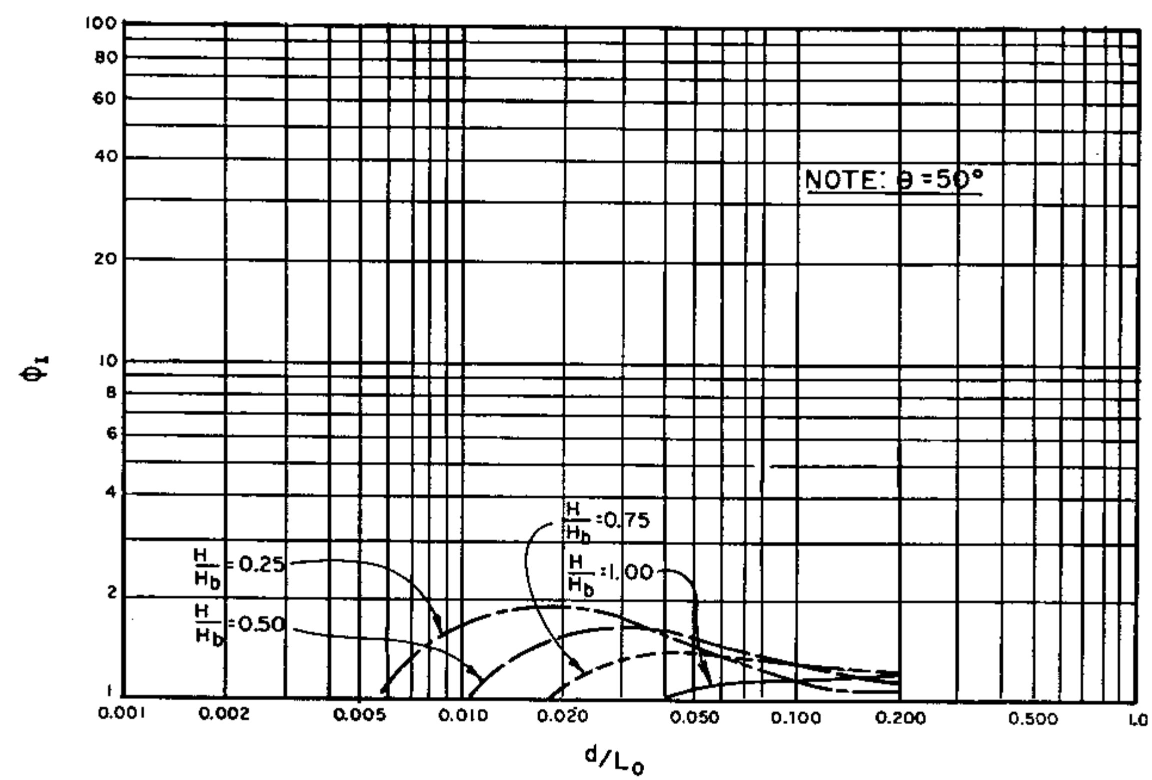


FIGURE 158
Nonlinear Inertial-Force Correction Factor, ϕ_I, as a Function of d/L_o and H/H_b for $\theta = 50°$

Function of d/LÚo¿ and H/HÚb¿ for [theta] = 50 deg.]

26.2-287

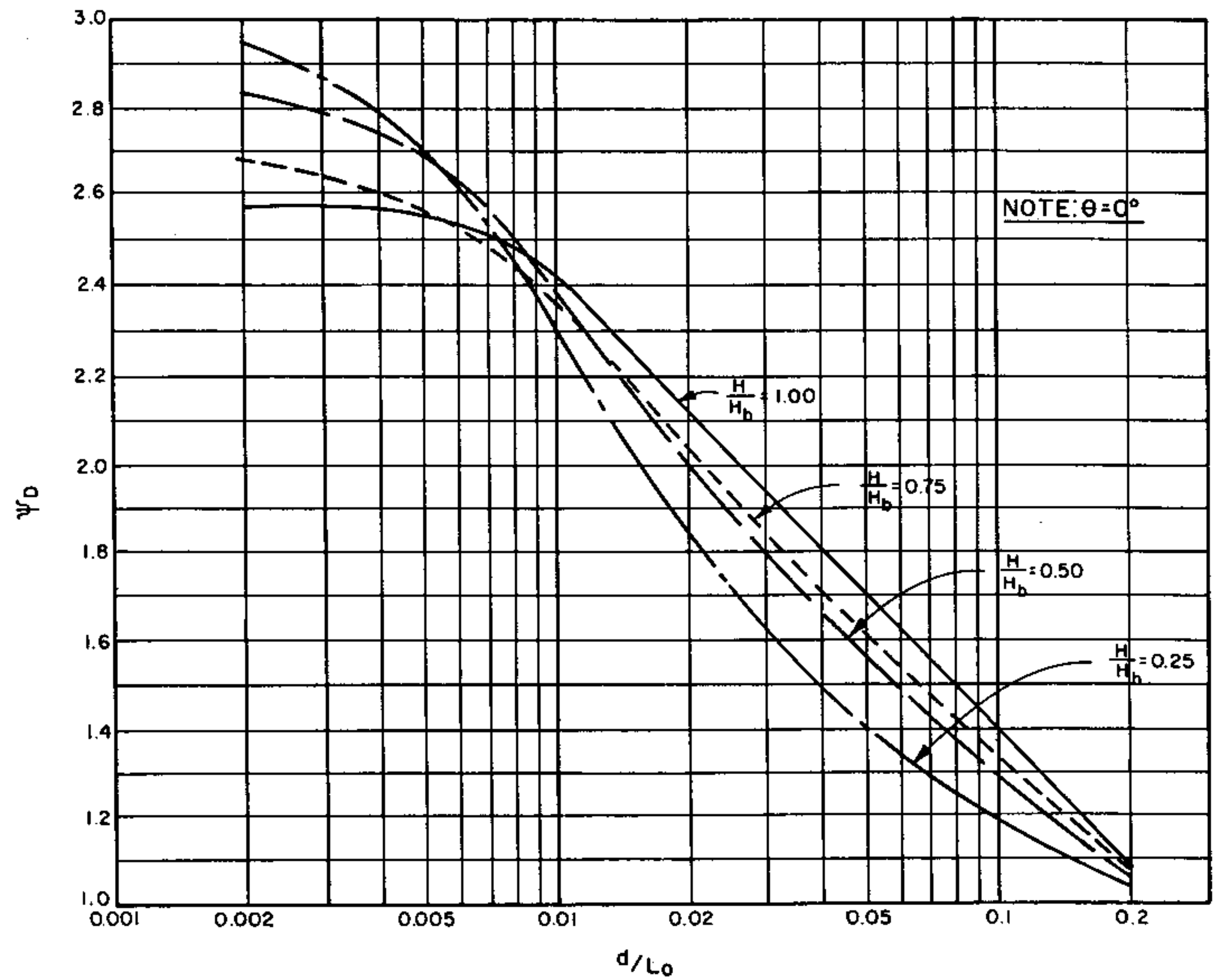


FIGURE 159
Nonlinear Drag-Moment Correction Factor, ψ_D, as a Function of d/L_o and H/H_b for $\theta = 0°$

Function of d/LÚo¿ and H/HÚb¿ for [theta] = 0 deg.]

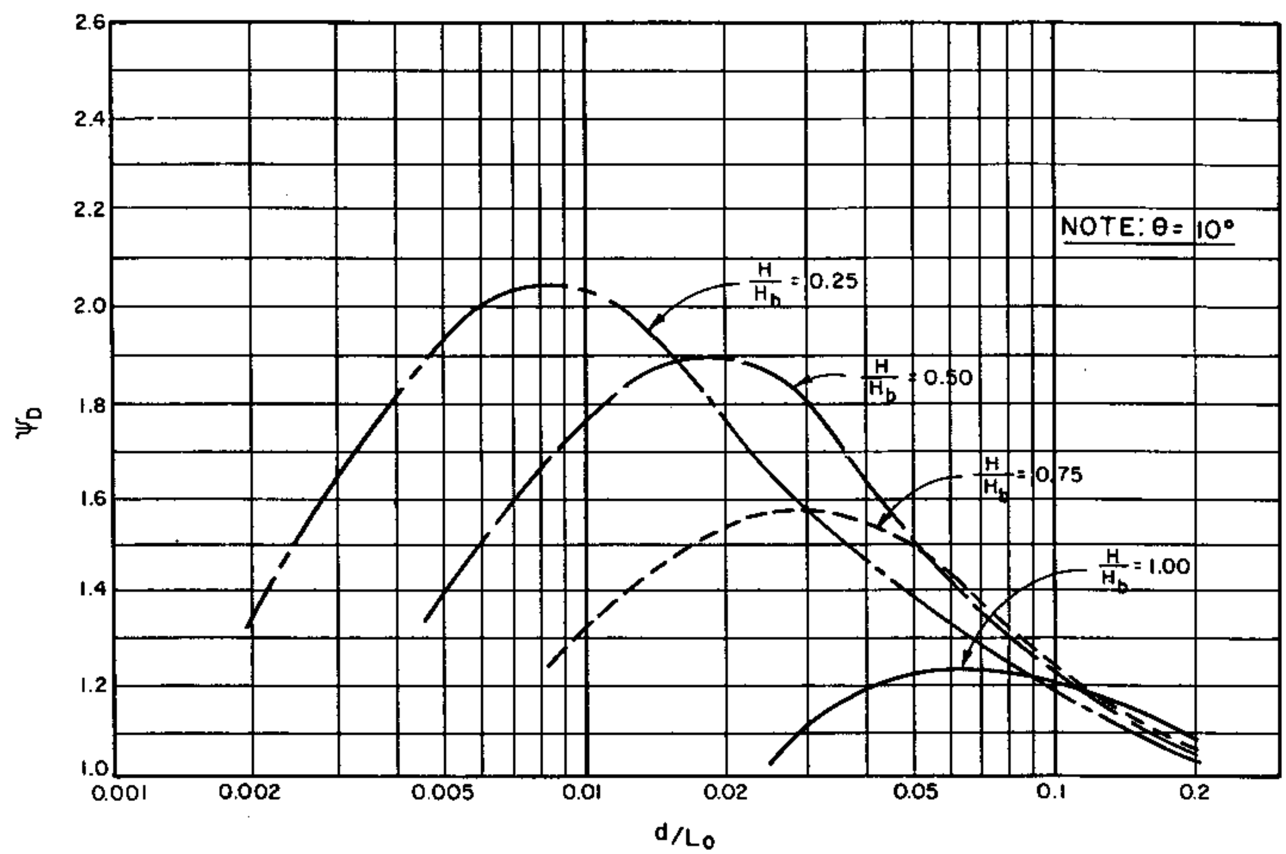


FIGURE 160
Nonlinear Drag-Moment Correction Factor, Ψ_D, as a Function of d/L_o and H/H_b for $\theta = 10°$

Function of d/LÚo¿ and H/HÚb¿ for [theta] = 10 deg.]

26.2-289

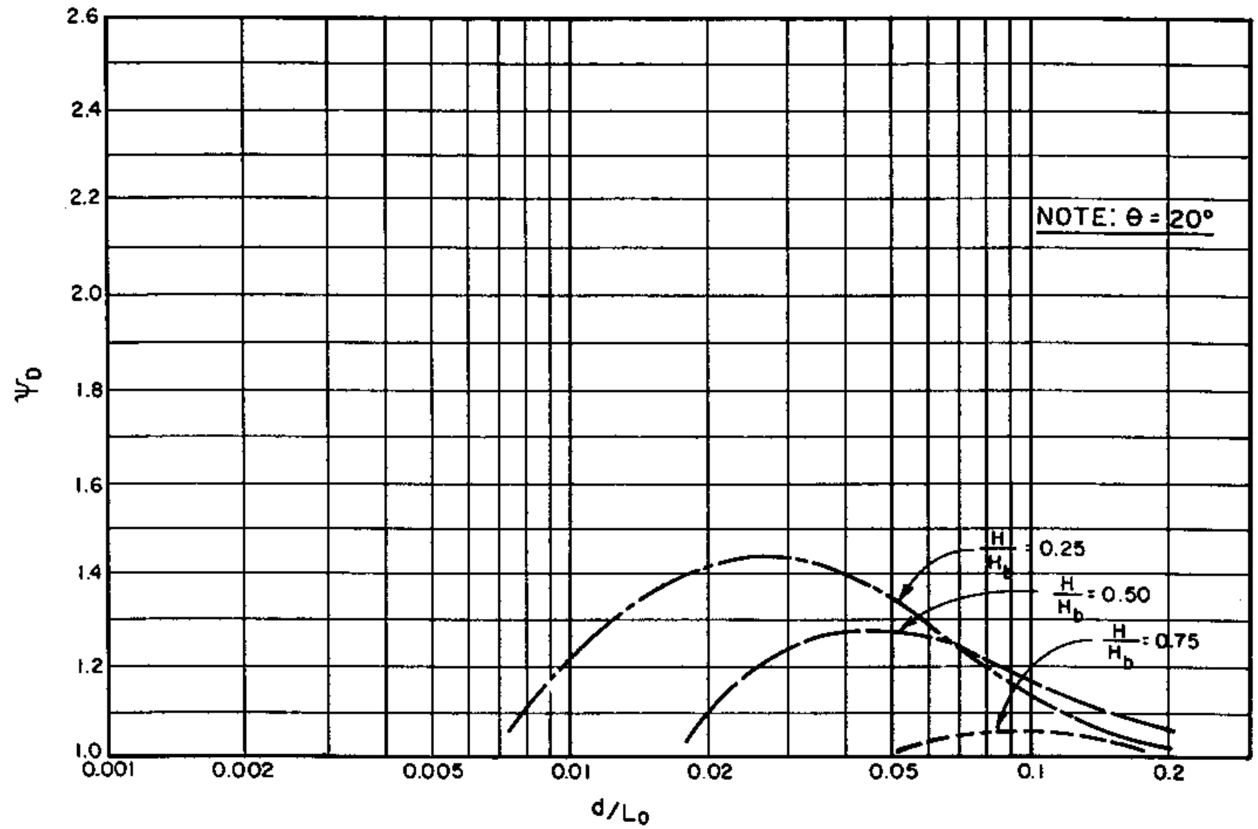


FIGURE 161
Nonlinear Drag-Moment Correction Factor, ψ_D, as a Function of d/L_o and H/H_b for θ = 20°

Function of d/LÚo¿ and H/HÚb¿ for [theta] = 20 deg.]

26.2-290

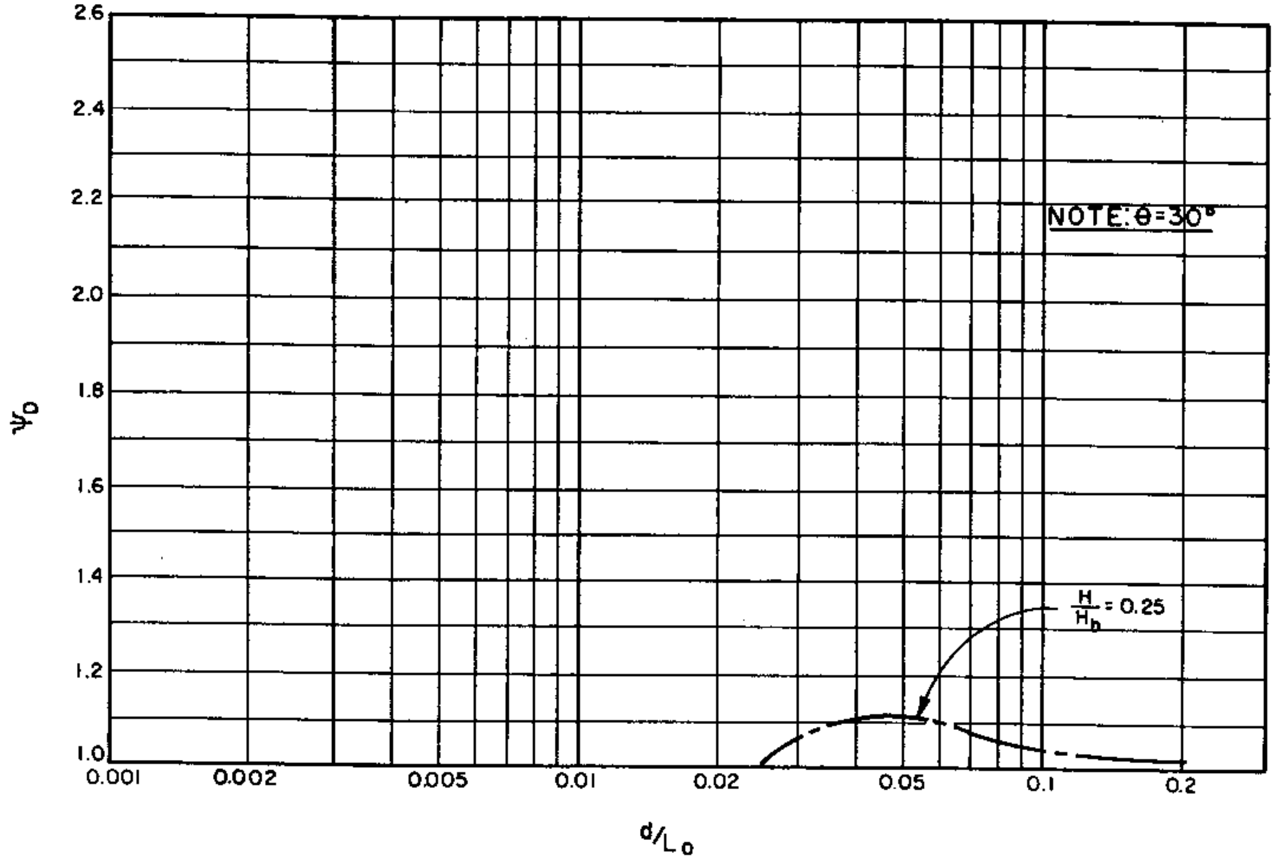


FIGURE 162
Nonlinear Drag-Moment Correction Factor, ψ_D, as a Function of d/L_o and H/H_b for $\theta = 30°$

Function of d/LÚo¿ and H/HÚb¿ for [theta] = 30 deg.]

$M_{IS_{\theta}}$ = inertial moment corrected for nonlinear effects

ψ_{I} = inertial-moment correction factor for nonlinear velocity and acceleration fields (found in Figures 163-166 as a function of d/L_{o}, H/H_{b}, and θ)

d. Total-Force and Moment. The total force on the pile at a given phase angle is given by:

$$F_{TS_{\theta}} = F_{DS_{\theta}} + F_{IS_{\theta}} \tag{7-31}$$

The total moment on the pile at a given phase angle is given by:

$$M_{TS_{\theta}} \; M_{DS_{\theta}} + M_{IS_{\theta}} \tag{7-32}$$

e. Maximum Values and Phase.

(1) Drag Force and Moment. The maximum drag force and moment, as well as the maximum nonlinear corrections, always occur under a wave crest; that is, when θ = 0 deg. Therefore, Figures 151 and 159, for which θ = 0 deg., yield well-defined nonlinear corrections as a function of d/L_{o} and H/H_{b} for the maximum drag force and moment, respectively. Drag components are symmetrical about θ = 0 deg.; therefore, force and moment for negative values of θ correspond to those for positive values of θ.

(2) Inertial Force and Moment. The maximum inertial force and the corresponding maximum inertial moment are complex functions of θ. These quantities are positive in front of the wave crest and negative behind it. However, the value of the wave-phase angle, θ, corresponding to the maximum inertial force and moment is unknown, a priori. Figures 167 and 168 give nonlinear correction factors, ϕ_{IM} and ψ_{IM} corresponding to the maximum values of inertial force and moment as a function of d/L_{o} and H/H_{b}. (Note: the values are not presented as a function of θ because the values of θ corresponding to the maxima are unknown.) These nonlinear corrections are given with respect to a standard reference level, namely $z = d$, and not with respect to the free surface, S_{θ}, as in Figures 151 to 166. As a result, these correction factors can only be used for uniform-diameter piles. The maximum value of inertial wave force, F_{mI} on a uniform-diameter pile is given by:

$$F_{mI} = \rho \, C_{M} \, \pi \left(\frac{D^{2}}{4} \right) \left(\frac{H}{T^{2}} \right) d \, (K_{IM})_{z=d} \, \phi_{IM} \tag{7-33}$$

WHERE: ρ = density of water

C_{M} = inertial, or added-mass, coefficient, obtained from Table 18

D = pile diameter

H = local wave height

T = wave period

d = water depth

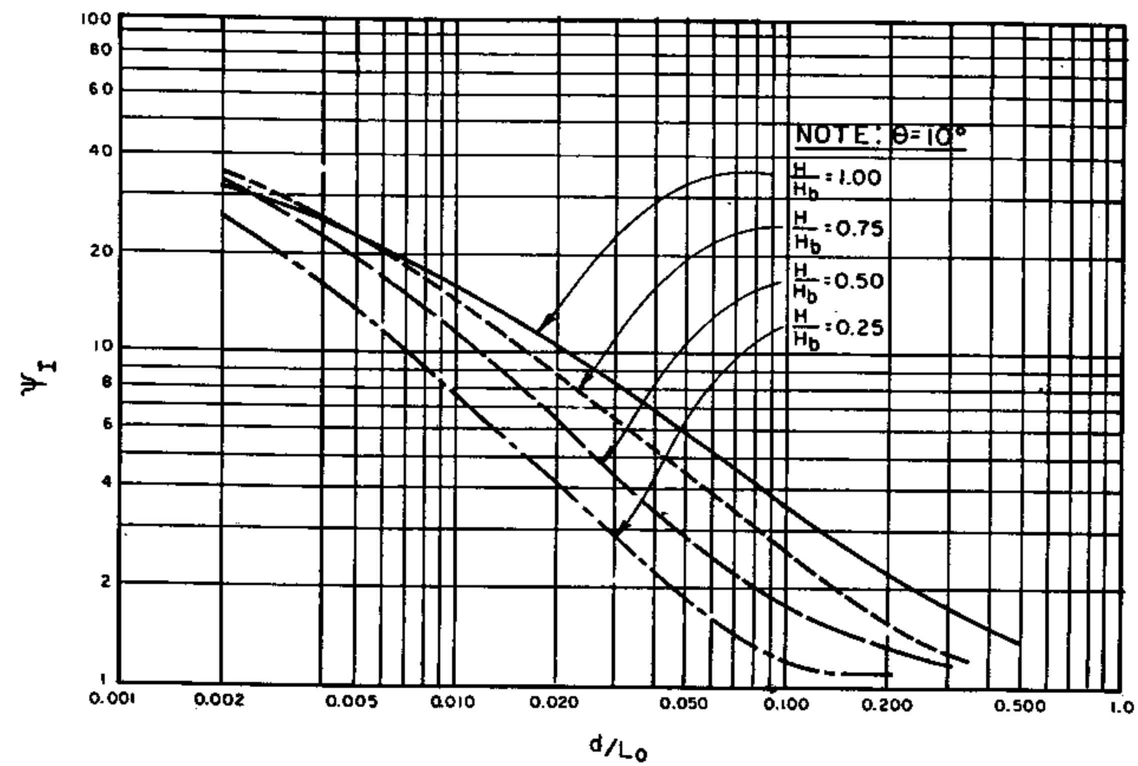


FIGURE 163
Nonlinear Inertial-Moment Correction Factor, ψ_I, as a Function of d/L_o and H/H_b for $\theta = 10°$

as a Function of d/LÚo¿ and H/HÚb¿ for [theta] = 10 deg.]

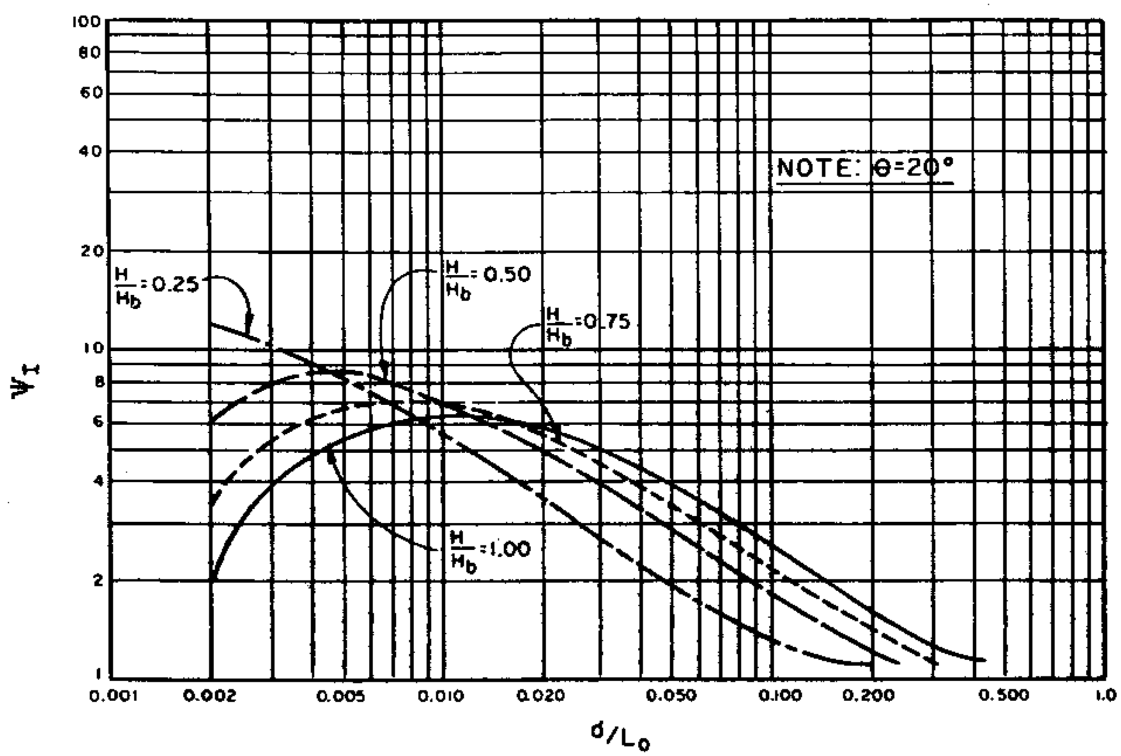


FIGURE 164
Nonlinear Inertial-Moment Correction Factor, Ψ_I, as a Function of d/L_o and H/H_b for $\theta = 20°$

as a Function of d/LÚo¿ and H/HÚb¿ for [theta] = 20 deg.]

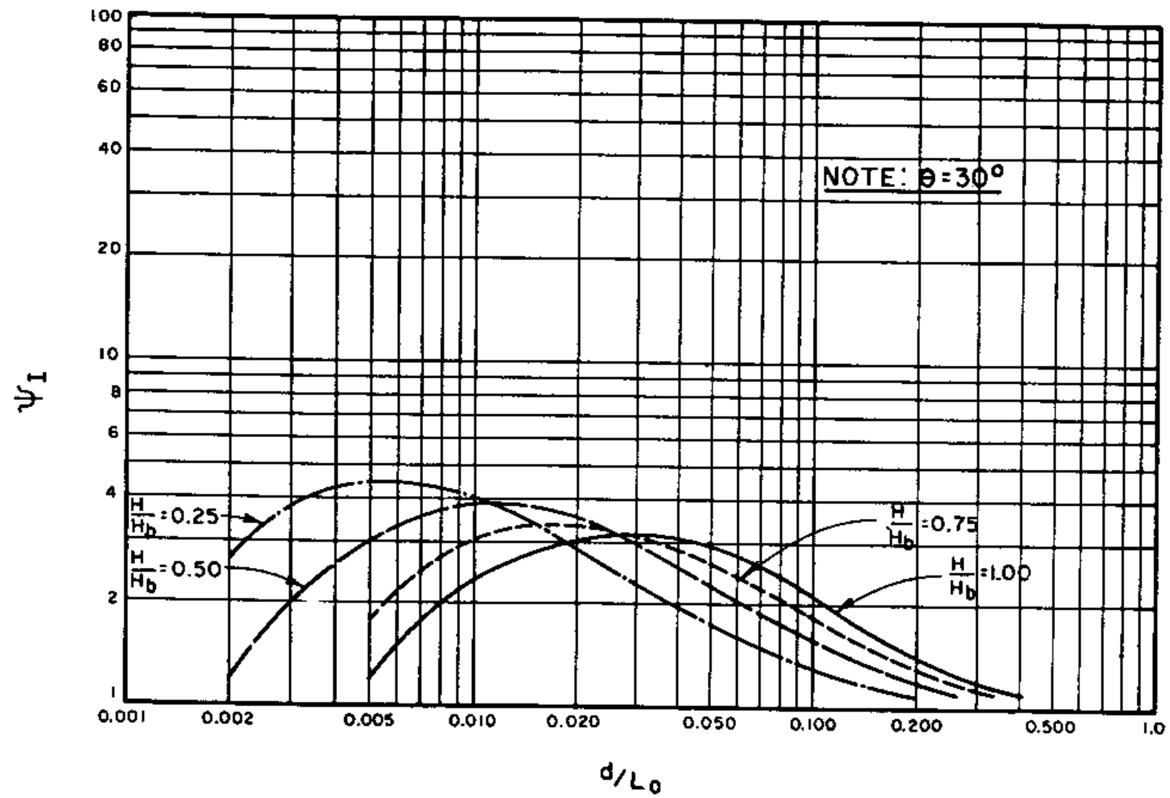


FIGURE 165
Nonlinear Inertial-Moment Correction Factor, Ψ_I, as a Function of d/L_o and H/H_b for $\theta = 30°$

as a Function of d/LÚo¿ and H/HÚb¿ for [theta] = 30 deg.]

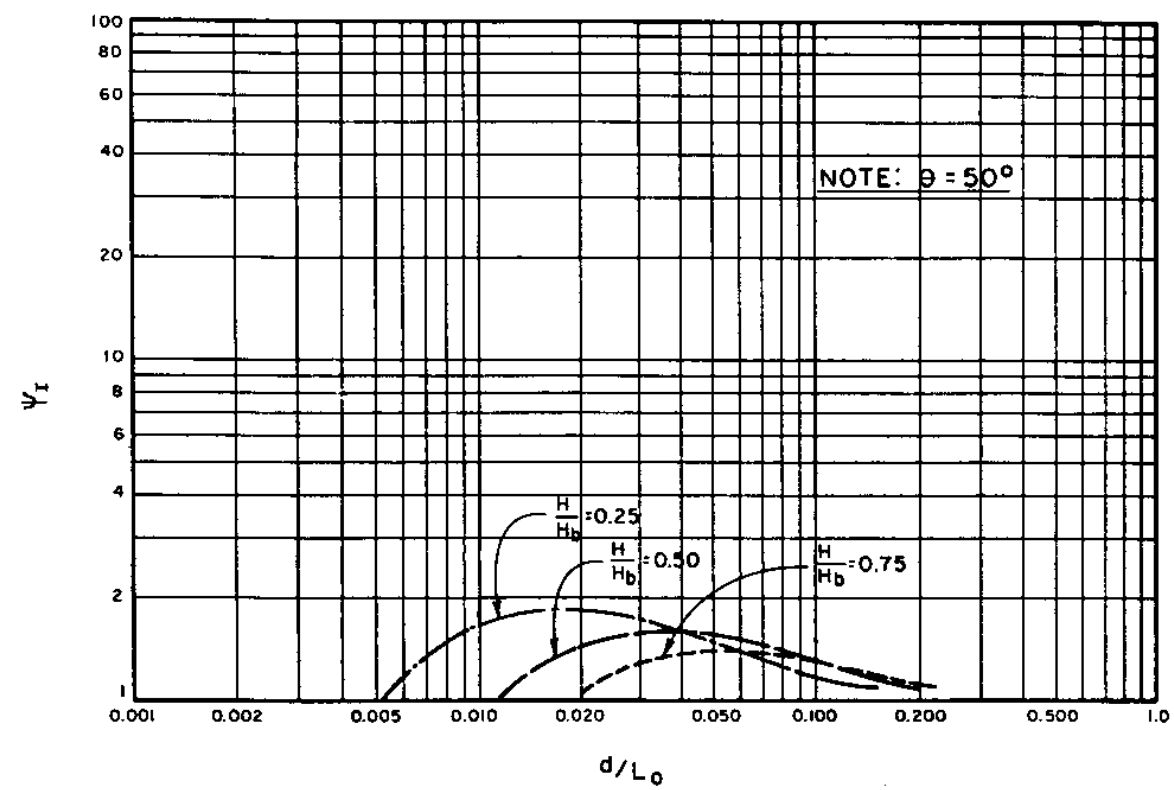


FIGURE 166
Nonlinear Inertial-Moment Correction Factor, Ψ_I, as a Function of d/L_o and H/H_b for θ = 50°

as a Function of d/LÚo¿ and H/HÚb¿ for [theta] = 50 deg.]

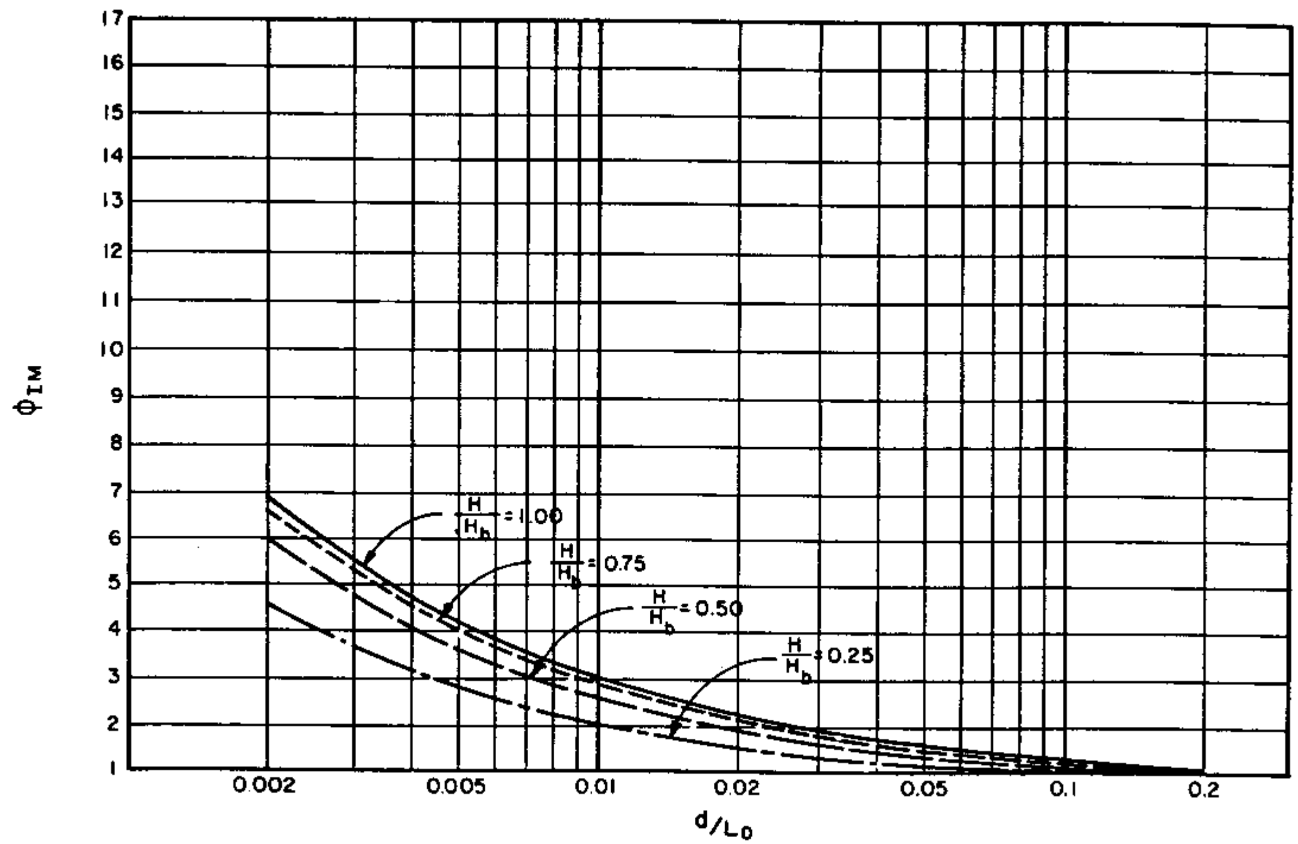


FIGURE 167

Nonlinear Correction Factor, ϕ_{IM}, for Determining Maximum Inertial Force, as a Function of d/L_o and H/H_b

for Determining Maximum Inertial Force, as a Function of d/LÚo¿ and H/HÚb¿

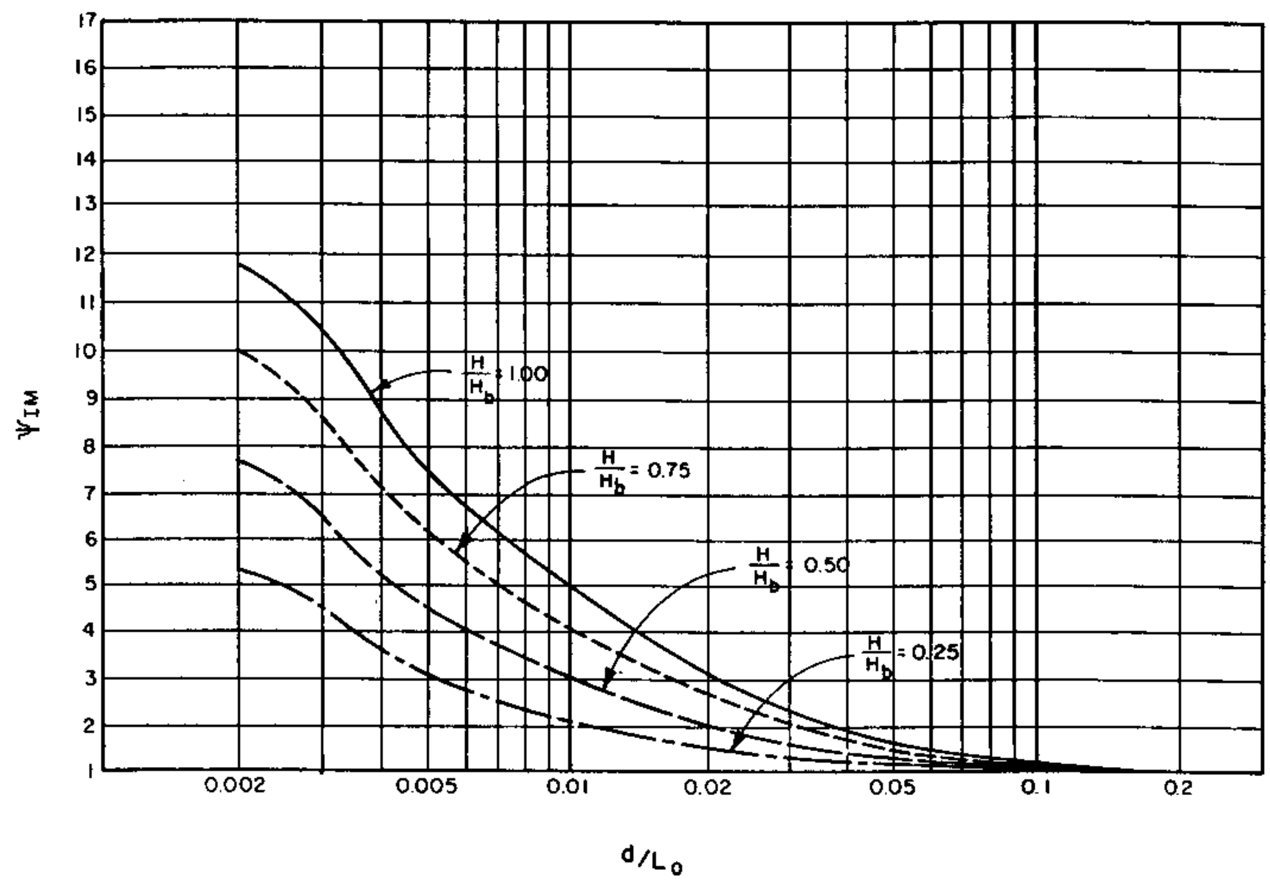


FIGURE 168
Nonlinear Correction Factor, ψ_{IM}, for Determining Maximum Inertial Force, as a Function of d/L_o and H/H_b

$(K_{IM})_{z=d}$ is found in Figure 147 as a function of z/d and d/L_o, where $z/d = 1$.

ϕ_{IM} is found in Figure 167 as a function of d/L_o and H/H_b.

The maximum value of inertial wave moment, M , on a uniform-diameter pile is given by:

$$M_{mI} = \rho\, C_m\, \pi \left(\frac{D^2}{4}\right) \left(\frac{H}{T^2}\right) d^2\, (\tau_{IM})_{z=d}\, \psi_{IM} \tag{7-34}$$

WHERE: $(\tau_{IM})_{z=d}$ is found in Figure 148 as a function of z/d and d/L_o, where $z/d = 1$

ψ_{IM} is found in Figure 168 as a function of d/L and H/H_b.

Drag and inertia add positively in front of the wave crest, and the maximum value for the total force and total moment occur in front of the wave crest. Inertia has a negative sign behind the wave crest; therefore inertial components must be subtracted from drag components behind the wave crest.

EXAMPLE PROBLEM 42

Given:
- a. Local wave height, $H = 15$ feet
- b. Water depth, $d = 20$ feet
- c. Wave period, $T = 10$ seconds
- d. Bottom slope is flat.
- e. $C_D = 0.7$ and $C_M = 1.6$
- f. Pile diameter is:

  $D_1 = 1.5$ feet for $z_o = 0$ foot to $z_1 = 18$ feet
  $D_2 = 2.0$ feet for $z_1 = 18$ feet to $z_2 = 24$ feet
  $D_3 = 1.5$ feet for $z_2 = 24$ feet to $z_3 = S_c$

Find: The force and moment acting as a function of wave-phase angle on a vertical pile.

Assume: $\rho = 2$ slugs per cubic foot

Solution: (1) Find L_o:

$$L_o = (g/2\pi)\, T^2 = (32.2/2\pi)\, (10)^2 = 512 \text{ feet}$$

(2) Determine the relative depth, d/L_o:

$d/L_o = 20/512 = 0.0391$; use $d/L_o = 0.039$

(3) Determine the breaking-wave height, H_b, and find H/H_b:

For a flat bottom slope: $H_b = 0.78\, d = 15.6$ feet

EXAMPLE PROBLEM 42 (Continued)

H/HÚb¿ = 15/15.6 = 0.962; use H/HÚb¿ = 0.96

H/HÚb¿ < 1

THEREFORE: The wave is nonbreaking.

(4) Determine the relative free surface, SÚ[theta]¿/d, for different phase angles, [theta]:

The SÚ[theta]¿/d values defining the position of the surface can be read from Figure 150; enter the d/LÚo¿ = 0.039.

At the point of intersection with the H/HÚb¿ curves encompassing the given value of H/HÚb¿ = 0.96 (in this case, H/HÚb¿ = 1.00 and H/HÚb¿ = 0.75), lines are drawn horizontally to the right until intersection with the corresponding [theta] curves. (Note each H/HÚb¿ curve has its own set of [theta] curves.)

Finally, vertical lines are drawn from the intersection points on the [theta] curves to give two values for S/d from which the correct value of S/d is found by interpolation. For example:

(a) When [theta] = 30 deg. and H/HÚb¿ = 1.00:

SÚ[theta]¿/d = 1.15

(b) When [theta] = 30 deg. and H/HÚb¿ = 0.75

SÚ[theta]¿/d = 1.17

(c) By interpolation:

When [theta] = 30 deg. and H/HÚb¿ = 0.96:

SÚ[theta]¿/d = 1.15

Table 19 was prepared following this procedure.

TABLE 19

Tabulation of SÚ[theta]¿/d Values for Example Problem 42

[theta] (degrees)...	0	10	20	30	50	75	100	130	180
H/HÚb¿									
1.00.	1.66	1.46	1.28	1.15	0.99	0.92	0.89	0.89	0.89
0.75.	1.47	1.40	1.27	1.17	1.02	0.93	0.90	0.89	0.89
0.96.	1.63	1.45	1.28	1.15	1.00	0.92	0.89	0.89	0.89

EXAMPLE PROBLEM 42 (Continued)

(5) The zÚi¿/d values for the changes in pile diameter are:

zÚ1¿   18
ÄÄÄÄÄ = ÄÄÄÄÄÄ = 0.90
  d   20

zÚ2¿   24
ÄÄÄÄÄ = ÄÄÄÄÄÄ = 1.20
  d   20

(6) Linear force and moment coefficients:

The linear forces coefficients, (KÚDM¿)Úz=z¿ and (KÚIM¿)Úz=z¿, and moment coefficients, ([tau]ÚDM¿)Úz=zÚi¿¿ and ([tau]ÚIM¿)Úz=z¿, are obtained from Figures 144, 147, 145, and 148, respectively, for the values of zÚi¿/d = zÚ1¿/d, zÚ2¿/d, and for all possible values of SÚ[theta]¿/d as a function of [theta]. These values are presented in Table 20.

(7) Drag force on a single pile:

For each [theta] value, the drag force is computed using Equation (7-18):

          1   H
FÚD¿ = (ÄÄÄ) [rho] CÚD¿ (ÄÄÄ)À2Ù d [(KÚDM¿)Úz=zÚ1¿¿ (DÚ1¿ - DÚ2¿)
          2   T

+ (KÚDM¿)Úz=zÚ2¿¿ (DÚ2¿ - DÚ3¿) + (KÚDM¿)Úz=SÚ[theta]¿¿ (DÚ3¿)]
cos [theta]³ cos [theta]³

For example, for [theta] = 0 deg.:

          1   15
FÚD¿ = (ÄÄÄ) (2) (0.7) (ÄÄÄÄ)À2Ù (20) [(33) (1.5 - 2.0
          2   10

+ (48)(2.0 - 1.5) + (71)(1.5)] (1) (1)

FÚD¿ = 3,591 pounds

For other [theta] values, the FÚD¿ is found by varying KÚDM¿ according to Table 20, and using this value in Equation (7-18). The value of the function cos [theta]³ cos [theta]³ will vary with [theta]. Values obtained for FÚD¿ for other values of are presented in Table 21.

(8) Inertial force on a single pile:

The inertial force on a single pile is computed using Equation (7-20):

[pi]   H

FÚI¿ = [rho] CÚM¿ (ÄÄÄÄ)(ÄÄÄÄ) d [(KÚIM¿)Úz=zÚ1¿¿ (DÚ1¿À2Ù - DÚ2¿À2Ù)
4 TÀ2Ù

+ (KÚIM¿)Úz=zÚ2¿¿ (DÚ2¿À2Ù - DÚ3¿À2Ù) + (KÚIM¿)Úz=SÚ[theta]¿¿ (DÚ3¿À2Ù)]
sin [theta]

26.2-301

TABLE 20
Linear Force and Moment Coefficients for Example Problem 42
(d/L_o = 0.039)

$\pm\theta$ (degrees)[3]	[1]	[2]	0	10	20	30	50	75	100	130	180
z_i/d or S_θ/d......	0.90	1.20	1.63	1.45	1.28	1.15	1.00	0.92	0.89	0.89	0.89
Coefficient											
	From Figure 144:										
$(K_{DM})_{z=z_i}$	33	48	71	60	50	44	37	34	33	33	33
	From Figure 147:										
$(K_{IM})_{z=z_i}$	35	48	0	59	51	46	39	36	34	34	0
	From Figure 145:										
$(\tau_{DM})_{z=z_i}$	16	30	67	48	36	28	19	17	16	16	16
	From Figure 148:										
$(\tau_{IM})_{z=z_i}$	17	30	0	45	34	28	19	18	16	16	0

[1] Values in this column are for z_i/d where i = 1.
[2] Values in this column are for z_i/d where i = 2.
[3] Values in third through ninth columns are for S_θ/d, where θ is from 0° to 180°.

TABLE 21
Linear Values of Force and Moment for Example Problem 42

θ (degrees)............	0	10	20	30	50	75	100	130	180
Force or Moment									
F_D (pounds)........	3,591	2,979	2,295	1,736	820	123	-54	-742	-1,796
F_I (pounds)........	0	204	355	476	638	756	737	573	0
F_T (pounds)........	3,591	3,183	2,650	2,212	1,458	879	683	-169	-1,796
M_D (foot-pounds)...	67,725	48,269	33,935	23,153	9,241	1,372	-589	-8,069	-19,530
M_I (foot-pounds)...	0	3,247	5,119	6,465	7,566	9,213	8,725	6,787	0
M_T (foot-pounds)...	67,725	51,516	39,054	29,618	16,807	10,585	8,136	-1,282	-19,530

EXAMPLE PROBLEM 42 (Continued)

[pi]   15
FÚI¿ = (2) (1.6) (ÄÄÄÄ) (ÄÄÄÄÄ) (20) {(35)[(1.5)À2Ù - (2.0)À2Ù]
4   10À2Ù

+ (48) [(2.0)À2Ù - (1.5)À2Ù] + (59) (1.5)À2Ù} (0.1736)

FÚI¿ = 204 pounds

The results for other values of [theta] are given in Table 21 for FÚI¿.

(9) Drag moment on a single pile:

The drag moment on a single pile is computed using Equation (7-22):

1   H
MÚD¿ = (ÄÄÄ) [rho] CÚD¿ (ÄÄÄ)À2Ù dÀ2Ù [([tau]ÚDM¿)Úz=zÚ1¿¿ (DÚ1¿ - DÚ2¿)
2   T

+ ([tau]ÚDM¿)Úz=zÚ2¿¿ (DÚ2¿ - DÚ3¿)

+ ([tau]ÚDM¿ Úz=SÚ[theta]¿¿ (DÚ3¿)] cos [theta]³ cos [theta]³

For example, for [theta] = 0 deg.

1   15
MÚD¿ = (ÄÄÄ) (2) (0.7) (ÄÄÄ)À2Ù 20À2Ù [(16) (1.5 - 2.0)
2   10

+ (30) (2.0 - 1.5) + (67) (1.5)] (1) (1)

MÚD¿ = 67,725 foot-pounds

The results for other values of [theta] are given in Table 21 for MÚD¿.

(10) Inertial moment on pile:

The inertial moment is computed using Equation (7-24):

[pi]   H
MÚI¿ = [rho] CÚM¿ (ÄÄÄÄ)(ÄÄÄÄ) dÀ2Ù [([tau]ÚIM¿)Úz=zÚ1¿¿ (DÚ1¿À2Ù - DÚ2¿À2Ù)
4   TÀ2Ù

+ ([tau]ÚIM¿)Úz=zÚ2¿¿ (DÚ2¿À2Ù - DÚ3¿À2Ù) + ([tau]ÚIM¿)Úz=SÚ[theta]¿¿

(DÚ3¿À2Ù)] sin [theta]

[pi]   15
MÚI¿ = (ÄÄÄÄ) (2) (1.6) (ÄÄÄÄÄ) (20)À2Ù {(17)[(1.5)À2Ù - (2.0)À2Ù]
4   10À2Ù

+ (30) [(2.0)À2Ù - (1.5)À2Ù] + (45) (1.5)À2Ù} (0.1736)

MÚI¿ = 3,246 foot-pounds

EXAMPLE PROBLEM 42 (Continued)

(11) Total forces and total moments are obtained using Equation (7-1) and a similar equation for moment:

$$F_T = F_D + F_I$$

$$M_T = M_D + M_I$$

for each value of [theta]. The corresponding results are found in Table 21 for F and M.

Recall that the values of F_D and M_I for the negative values of [theta] (-180 deg. < / = [theta] < / = 0 deg.) are identical to the values of F_D and M_D for the respective positive values of [theta] (0 < / = [theta] < / = 180 deg.); that is:

F_D for [theta] = F_D for -[theta]

M_D for [theta] = M_D for -[theta[

whereas the values of F_I and M_I for negative values of [theta] are of opposite sign from the values of F_I and M_I for the respective positive values of [theta]; that is:

F_I for [theta] = $-F_D$ for -[theta]

M_I for [theta] = $-M_D$ for -[theta]

Therefore, Table 21 can easily be computed for the entire cycle of [theta] (-180 deg. < / = [theta] < / = +180 deg.).

(12) Maximum values--linear theory:

The maximum total force and maximum total moment on a single pile according to linear theory are seen from Table 21 to occur when [theta] [approx =] 0 deg., which is the value of [theta] when the wave crest is at the structure:

F_m = 3,591 pounds

M_m = 67,725 foot-pounds

The lever arm at the maxima linear force and moment is:

$$z_{mDI} = \frac{M_{mDI}}{F_{mDI}} = \frac{67{,}725}{3{,}591} = 18.9 \text{ feet}$$

(13) Nonlinear correction due to velocity and acceleration field:

The nonlinear correction factor, $[phi]_D$, for the drag forces is given by Figures 151 to 154 and presented as a function of [theta] in Table 22.

EXAMPLE PROBLEM 42 (Continued)

The nonlinear correction factor, [phi]ÚI¿ for the inertial forces is given by Figures 155 to 158 and presented as a function of [theta] in Table 22.

The nonlinear correction factor, [psi]ÚD¿ for the drag moments is given by Figures 159 to 162 and presented as a function of [theta] in Table 22.

The nonlinear correction factor, [psi]ÚI¿, for the inertial moments is given by Figures 163 to ~66 and presented as a function of [theta] in Table 22.

TABLE 22
Nonlinear Correction Factors for Example Problem 42
(d/LÚo¿ = 0. 039)

+/-[theta] (degrees). . .					
Correction Factor					
[phi]ÚD¿.	1.56	1.15	1	1	1
[phi]ÚI¿.	....	5.26	3.79	2.68	1
[psi]ÚD¿.	1.80	1.25	1	1	1
[psi]ÚI¿.	....	6.73	4.40	3.10	1

The nonlinear corrections apply only for:

0 deg. < / = [theta] 30 deg. for drag

0 deg. < / = [theta] 50 deg. for inertial

For values of wave-phase angle outside of these values, the linear theory gives larger values of force and moment than nonlinear theory and therefore should prevail as a conservative estimate.

The nonlinear corrections are identical for [theta] and -[theta] values. However, when [theta] is negative, the inertial forces and moments are subtracted from the drag forces and moments. Multiplying the correction factors, [phi]ÚD¿ or ÚI¿ and [psi]ÚD¿ or ÚI¿, by the corresponding forces and moments given by Table 21 (Equations (7-27)-(7-30)) yields the nonlinear values for force and moment, which exceed the linear results.

EXAMPLE PROBLEM 42 (Continued)

The nonlinear results (of applying the correction factors given in Table 22 to the values in Table 21) are given in Table 23.

(14) Total forces and moments for positive and negative values of [theta]:

Adding $F_{DS\theta}$ and $F_{IS\theta}$ and $M_{DS\theta}$ and $M_{IS\theta}$ gives the total values of the force, F, and moment, M (Equations (7-31) and (7-32)). The corresponding results for 0 deg. < / = [theta] < / = 180 deg. are shown in Table 23. For -180 deg. < / = [theta] < / = 0 deg., the inertial forces and moments are subtracted from the drag forces and moments, respectively, to obtain the total values for force and moment, respectively. The results of this calculation are also shown in Table 23. Recall that a positive value of force and moment is in the direction of wave travel and that a negative value is in the opposite direction.

Figures 169 and 170 present the final result of force and moment on the single pile of this example problem. It is seen that the maximum force occurs when [theta] = 2 deg. and is equal to F_m = 5,650 pounds. The maximum moment also occurs when [theta] = 2 deg. and is equal to M_m = 123,450 foot-pounds.

(15) The lever arm at the maxima nonlinear force and moment is:

$$z_{mDI} = \frac{M_{mDI}}{F_{mDI}} = \frac{123,450}{5,650} = 21.8 \text{ feet}$$

10. CASE 6--FORCES AND MOMENTS ()N A COMBINATION OF PILES. Wave forces on a combination of piles supporting a structure are calculated by plotting the forces on a single pile as a function of phase angle. The combination of piles is then superimposed over the plot of wave forces, and the maximum force for the system is found by summing the forces on the individual piles for small changes in phase angle, [theta]. The maximum force generally occurs when the central pile is located near the wave crest. A similar procedure is used to determine the maximum moment. The procedure is illustrated by Example Problem 43. The following equation is used to determine the phase difference between piles:

$$\Delta\theta = \left(\frac{2\pi}{L} \right) (\Delta x) \qquad (7\text{-}35)$$

WHERE: [DELTA][theta] = phase difference between the piles

L = wavelength

x = spacing between the piles in the direction of wave advance (that is, along the x-coordinate axis)

TABLE 23
Forces and Moments for Phase Angles for Example Problem 42
After Nonlinear Corrections

θ (degrees).........	0	10	20	30	50	75	100	130	180
Force or Moment									
F_{DS}...........	5,602	3,426	2,295	1,736	820	123	-54	-742	-1,796
F_{IS}...........	0	1,073	1,346	1,276	638	756	737	573	0
F_T............	5,602	4,499	3,641	3,012	1,458	879	683	-169	-1,796
M_{DS}...........	121,905	60,336	33,935	23,153	9,241	1,372	-589	-8,069	-19,530
M_{IS}...........	0	21,852	22,524	20,042	7,566	9,213	8,725	6,787	0
M_T............	121,905	82,188	56,459	43,195	16,807	10,585	8,136	-1,282	-19,530
θ (degrees).........	**0**	**-10**	**-20**	**-30**	**-50**	**-75**	**-100**	**-130**	**-180**
Force or Moment									
F_T............	5,602	2,353	949	460	182	-633	-791	-1,315	-1,796
M_T............	121,905	38,484	11,411	3,111	1,675	-7,841	-9,314	-14,856	-19,530

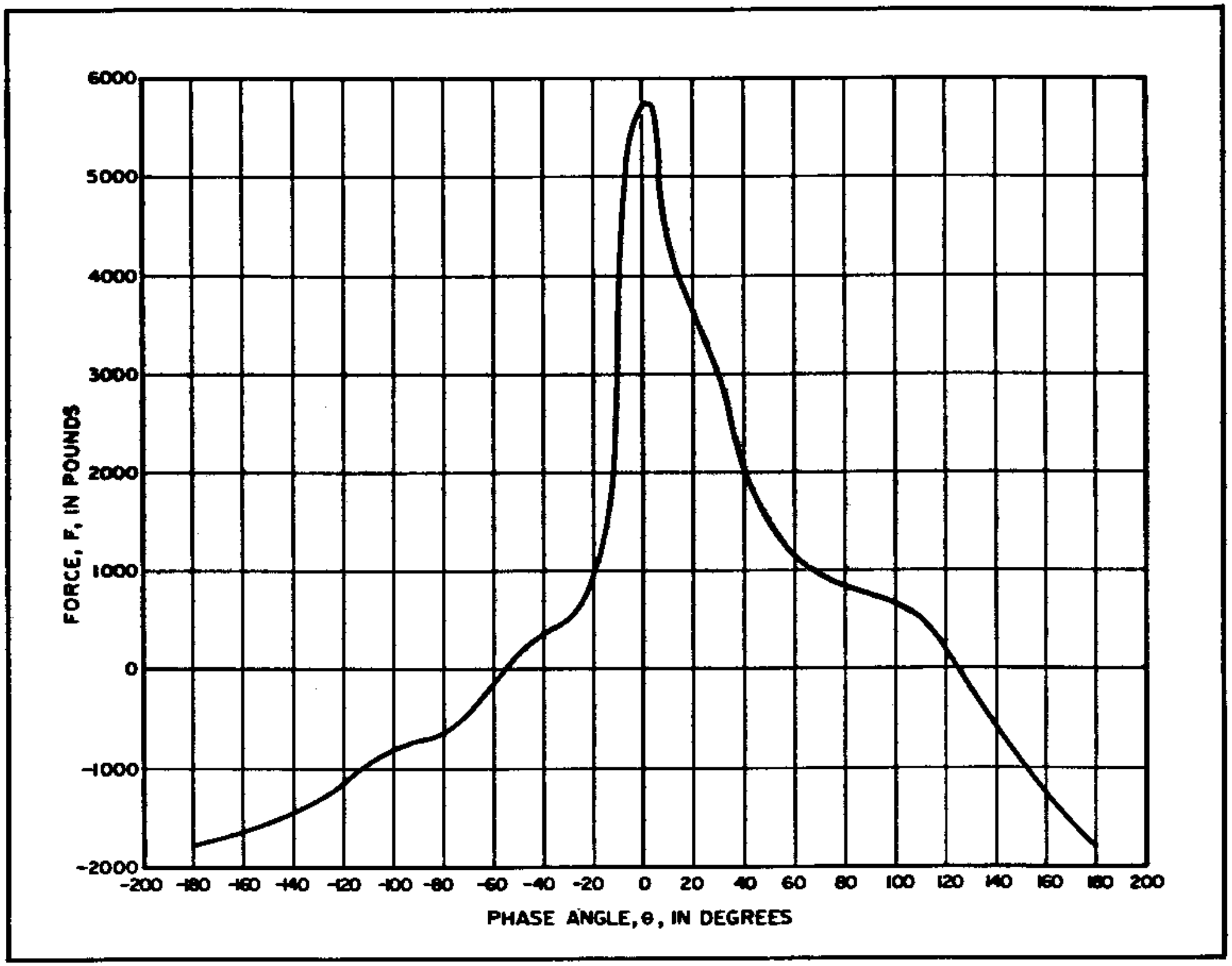


FIGURE 169
Results of Example Problem 42: Force

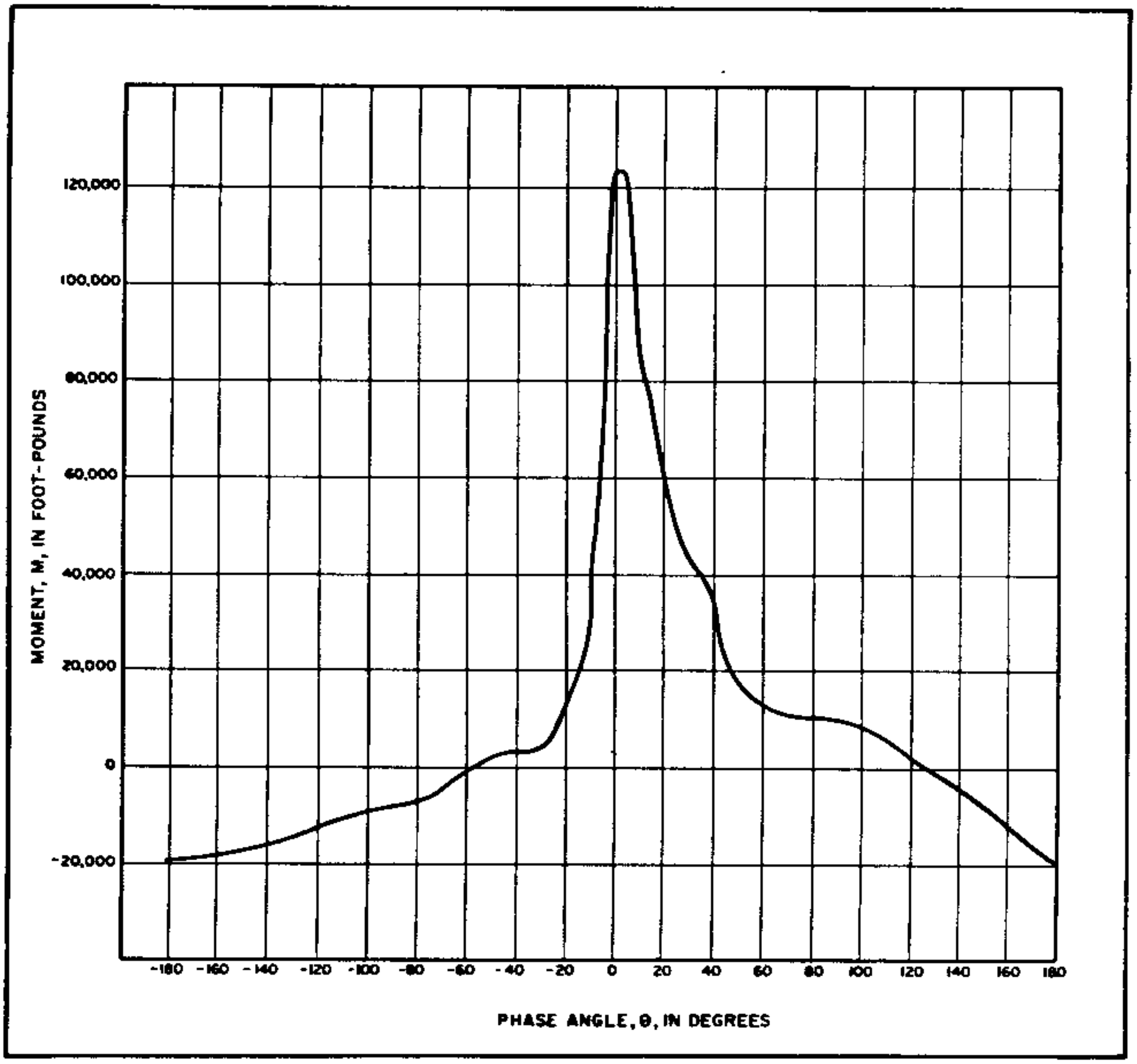


FIGURE 170
Results of Example Problem 42: Moment

EXAMPLE PROBLEM 43

Given: Five identical piles, identical also to the one given in the previous example problem, separated by a distance of $\Delta x = 16$ feet and structurally connected.

Find: The maximum force and moment on the combination of piles.

Solution: (1) Find L:

From Example Problem 42, $d/L_o = 0.039$

From Figure 2 for $d/L_o = 0.039$:

$d/L = 0.082$

$$L = \frac{d}{0.082} = \frac{20}{0.082}$$

$L = 244$ feet

(2) Find F and M:

The phase difference between piles is given by Equation (7-35):

$$\Delta\theta = \left(\frac{2\pi}{L}\right)(\Delta x) = \left(\frac{360}{243}\right)(16) = 23.7 \text{ deg.}$$

Because the maximum forces generally occur when the central pile is located near the wave crest, use Pile 3 as the reference pile, at which x = 0 (see Table 24).

Referring to Figures 169 and 170, one shifts the location of the pile group with respect to θ by 2-degree increments. The results of this procedure are shown in Table 24, which indicates that the maximum force and moment occur when the middle pile is about 2 deg. ahead of the wave crest. Therefore, the maximum force on the total piling system is:

F = 11,495 pounds

and the sum of all moments (not to be taken as the total moment) is:

M = 206,750 foot-pounds

11. CASE 7--BRACING.

a. General. The wave force on a bracing beam between two vertical piles is determined by the same method as in the case of a vertical pile; that is, the wave force per unit length, Δ_z, is also the sum of a drag force per unit length of pile, f_D, and an inertial force per unit length of pile, f_I. However, bracings are generally beams of small diameter such that only

TABLE 24
Forces and Moments for Example Problem 43

Pile Number......	1	2	3	4	5	Total Acting on Pile Group
θ (degrees)...	-47.20	-23.70	0	23.70	47.40	
Force.........	208	750	5,602	3,290	1,625	11,475 lb
Moment........	2,500	7,500	123,100	50,000	19,500	202,600 ft-lb
θ (degrees)...	-45.20	-21.70	2	25.70	49.40	
Force.........	270	825	5,650	3,208	1,542	11,595 lb
Moment........	3,000	11,500	123,900	48,000	18,100	204,500 ft-lb
θ (degrees)...	-43.20	-19.70	4	27.70	51.40	
Force.........	290	917	5,646	3,063	1,438	11,354 lb
Moment........	3,500	13,000	123,100	45,000	16,800	201,400 ft-lb

the drag force is of importance. In addition, a horizontal beam parallel to the wave crest in and out of the water is subjected to a wave-slamming force. This subject is not presented herewith because such design is not recommended.

The local force per unit length of bracing is a function of the distance, z_B of the beam above the bottom; if the beam is at an angle with the wave crest, this force is also a function of the wave-phase angle, θ.

b. Horizontal Bracing.

(1) Force. In order to determine the wave force on a horizontal beam of diameter, D, at relative distance above the bottom, z_B/d, first determine the wave forces at distances above the bottom of:

$$z_1/d = z_B/d - D/(2\,d) \qquad (7\text{-}36)$$

and

$$z_2/d = z_B/d + D/(2\,d) \qquad (7\text{-}37)$$

WHERE: z_i = distance above bottom of point i

d = water depth

z_B = distance of center of beam above bottom

D = beam diameter

From Figure 144, find $(K_{DM})_{z1}$ and $(K_{DM})_{z2}$ for z_1/d and z_2/d. The force per unit length of bracing, F_B, is then determined as follows:

$$F_B \text{ per unit length of bracing} = (F)_{z_2} - (F)_{z_1} \qquad (7\text{-}38)$$

WHERE: $(F)_{z_i} = \left(\frac{1}{2}\right) \rho\, C_D \left(\frac{H}{T}\right)^2 d\, (K_{DM})_{z=z_i}$ = force per unit length of bracing at an arbitrary distance above bottom, z_i (7-39)

z_i = distance above bottom of point i

ρ = density of water

C_D = drag coefficient (obtained from Table 17)

H = local wave height

T = wave period

d = water depth

$(K_{DM})_{z=z_i}$ = is obtained from Figure 144 as a function of z/d and d/L_o.

(2) Moment. The corresponding moment, M_B, about the mudline is:

$$M_B = (F_B \text{ per unit length of bracing})(z_B) \qquad (7\text{-}40)$$

WHERE: z_B = distance of center of beam above bottom

c. Angle Bracing. If the beam is at an angle with the vertical, one must initially determine the force on each pile element of unit length as a function of elevation. If the beam is at an angle with the wave direction, the phase angle, [theta], becomes an additional parameter.

All these calculations are based on the linear wave theory. Therefore, appropriate safety margins are recommended to account for nonlinear effects, particularly when determining the force near the free surface.

12. CASE 8--FORCES DUE TO BREAKING WAVES. Just prior to the inception of wave breaking, a wave traveling on a slope peaks up to a limiting value, H_b, as described in Subsection 6.3., LIMITING WAVE HEIGHT.

In shallow water, near breaking, the drag force is the primary component of wave force on piles. Both small and full-scale laboratory experiments indicate that breaking waves on rigidly supported piles induce a short-duration "slamming" force caused by the impact of the breaking wave on the pile (Hall (1958) and Ross (1959)). This slamming force (analogous to the dynamic component of force on walls subjected to breaking waves) is superimposed on a wave force similar to that for a nonbreaking wave. The slamming force is of short duration ($<$ 0.1 second) and occurs over a small portion of the wave profile. Furthermore, prototype piles have elasticity, which may not have been properly scaled in the model tests. The elastic properties of the pile can absorb very short-duration loads, as is evidenced by numerous prototype installations located in the surf zone which have performed adequately without taking the slamming force into account. The slamming force is usually of such magnitude that the sizing of structural members may lead to impractical designs if design criteria were to include the slamming force.

The recommended procedure for determining breaking-wave forces and moments on piles is to use the procedure outlined in Subsection 7.7, CASE 3, based solely on drag, for $H/H_b = 1$ and a drag coefficient of $C_D = 1.0$. For situations where the wave slamming force is believed to be critical to design, force and moment may be conservatively estimated by multiplying the values obtained in the above procedure by two.

13. METRIC EQUIVALENCE CHART. The following metric equivalents were developed in accordance with ASTM E-621. These units are listed in the sequence in which they appear in the text of Section 7. Conversions are approximate.

32.2 feet per second^2 = 9.81 meters per second^2

1 x 10^{-5} feet^2 per second = 9.29 x 10^{-7} meters per second

1 foot = 30.5 centimeters

REFERENCES

Shore Protection Manual, U.S. Army Coastal Engineering Research Center, 3d ed., Vols. I, II, and III, Stock No. 008-022-00113-1, U.S. Government Printing Office, Washington, DC, 1977.

Wiegel, R.L.; "Oscillatory Waves," Bulletin, Special Issue No. 1, U.S. Army, Beach Erosion Board, July 1948.

Wiegel, R.L.; "Diffraction of Waves by a Semi-Infinite Breakwater," Journal of the Hydraulics Division, ASCE, Vol. 88, HY 1, January 1962, pp. 27-44.

Johnson, J.W.; "Generalized Wave Diffraction Diagrams," Proceedings of Second Conference on Coastal Engineering, Houston, Texas, November 1951, Council on Wave Research, The Engineering Foundation, Berkely, CA, 1952.

Ippen, Arthur T., ed.; Estuary and Coastline Hydrodynamics, McGraw-Hill Book Company, New York, 1966.

Goda, Y.; "A Synthesis of Breaker Indices," Transactions of the Japanese Society of Engineers, Vol. 2, Pt. 2, 1970.

Weggel, J.R.; "Maximum Breaker Height," Journal of the Waterways, Harbors and Coastal Engineering Division, ASCE, Vol. 98, No. WW4, Paper 9384, 1972.

ASTM E-621; "Standard Practice for the Use of Metrics (SI) Units in Building Design and Construction," Annual Book of ASTM Standards, Part 18, American Society for Testing and Materials (ASTM), Philadelphia, PA, 1979.

Bretschneider, Charles L.; "Overwater Wind and Wind Forces," Handbook of Ocean and Underwater Engineering, McGraw-Hill Book Company, New York, 1969, pp. 12-2 - 12-24. (Editors: John J. Myers, Carl H. Holm, and Raymond F. McAllister)

U.S. Army Corps of Engineers; "Method of Determining Adjusted Windspeed, UÚA¿, for Wave Forecasting," Coastal Engineering Technical Note, CETN-I-5, Coastal Engineering Research Center, Fort Belvoir, VA, 1981a.

Resio, D.T., and Vincent, C.L.; "Estimation of Winds Over the Great Lakes," MP H-76-12, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS, June 1976 (Final Report).

U.S. Army Corps of Engineers; "Revised Method for Wave Forecasting in Deep Water," Coastal Engineering Technical Note, CETN-I-7, Coastal Engineering Research Center, Fort Belvoir, VA, 1981b.

U.S. Army Corps of Engineers; "Revised Method for Wave Forecasting in Shallow Water," Coastal Engineering Technical Note, CETN-I-6, Coastal Engineering Research Center, Fort Belvoir, VA 1981c.

Summary of Synoptic Meteorological Observations, prepared under the direction of the U.S. Naval Weather Service Command by the National Climatic Center, Asheville, NC. (Copies are obtainable from the National Technical Information Service, Springfield, VA 22161.)

Hogben, N., and Lumb, F.E.; Ocean Wave Statistics, Ministry of Technology, National Physical Laboratory, Her Majesty's Stationery Office, London, 1967.

Coastal Zone Management (CZM) Act of 1972, PL 92-583.

DOD Instruction 4165.59 of 29 December 1975.

Stoa, Philip N.; "Wave Runup on Rough Slopes," Coastal Engineering Technical Aid (CETA) No. 79-1, U. S. Army, Corps of Engineers, Coastal Engineering Research Center, Fort Belvoir, VA, July 1979.

Stoa, Philip N.; "Revised Wave Runup Curves for Smooth Slopes," Coastal Engineering Technical Aid (CETA) No. 78-2, U.S. Army, Corps of Engineers, Coastal Engineering Research Center, Fort Belvoir, VA, July 1978.

Saville, T., Jr.; "Wave Runup on Shore Structures," Journal of the Waterways and Harbors Division, ASCE, Vol. 82, No. WW2, 1956.

Seelig, William N.; "Two-Dimensional Tests of Wave Transmission and Reflection Characteristics of Laboratory Breakwaters, " Technical Report No. 80-1, U.S. Army, Corps of Engineers, Coastal Engineering Research Center, Fort Belvoir, VA, June 1980.

Wiegel, Robert L.; "Transmission of Waves Past a Rigid Vertical Thin Barrier," Journal of the Waterways and Harbors Division, ASCE, Vol. 86, No. WWI, March 1960, pp. 1-12.

Perroud, Paul Henri; "The Solitary Wave Reflection Along a Straight Vertical Wall at Oblique Incident," Ph.D. Thesis, University of California; also, Inst. Eng. Res., Tech. Rept. No. 99-3, September 1957 (Unpublished).

Seelig, William N., and Ahrens, John P.; "Estimation of Wave Reflection and Energy Dissipation Coefficients for Beaches, Revetments and Breakwaters," Unpublished Draft, U.S. Army Coastal Engineering Research Center, Fort Belvoir, VA, 18 August 1980.

Walker, James R., Palmer, Robert Q., and Dunham, James W.; "Breakwater Back Slope Stability," Proceedings, Civil Engineering in the Oceans/III, 9-12 June 1975, Vol. 2, ASCE, New York, 1975, pp. 879-898.

El Ghamry, Osman A.; "Wave Forces on a Dock," Hydraulic Engineer Laboratory, Wave Research Projects, University of California, Institute of Engineering Research, Technical Report HEL-9-1, Berkeley, CA, October 1963 (Submitted under contract with the Beach Erosion Board, Corps of Engineers, U. S. Army).

French, Jonathan A.; "Wave Uplift Pressures on Horizontal Platforms," W.M. Keck Laboratory of Hydraulics and Water Resources, Division of Engineering and Applied Science, California Institute of Technology, Pasadena, CA, Report No. KH-R-19, July 1969.

Wang, Hsiang; "Estimating Wave Pressures on a Horizontal Pier," Technical Report No. R 546, Naval Civil Engineering Laboratory, Port Hueneme, CA, October 1967.

Richey, E.P., and Nece, R.E.; "Floating Breakwaters--State of the Art," Proceedings, 1974 Floating Breakwaters Conference Papers, Newport, RI, University of Rhode Island Marine Technical Report Series No. 24, Kingston, RI, April 1974, pp. 1-20.

Harms, Volker W.; "Floating Breakwater Performance Comparison," 17th International Conference on Coastal Engineering, Abstracts-in-Depth, Sydney, Australia, 23-28 March 1980, The Institution of Engineers, Australia, Barton, A.C.T., Australia, pp. 328-329.

Kowalski, T., ed.; Proceedings, 1974 Floating Breakwaters Conference Papers, Newport, RI, University of Rhode Island Marine Technical Report Series No. 24, Kingston, RI, April 1974.

Hales, Lyndell Z.; "Floating Breakwater State-of-the-Art Literature Review," Unpublished Draft, Hydraulics Laboratory, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS, September 1980. (Prepared for U.S. Army Engineer Coastal Engineering Research Center, Fort Belvoir, VA.)

Harms, Volker W., and Bender, Thomas J.; "Preliminary Report on the Application of Floating-Tire-Breakwater Design Data," Water Resources and Environmental Engineering Research Report No. 78-1, Department of Civil Engineering, State University of New York at Buffalo, NY, April 1978.

Seymour, R.J., and Isaacs, J.D.; "Tethered Float Breakwaters," Proceedings, 1974 Floating Breakwaters Conference Papers, Newport, RI, University of Rhode Island Marine Technical Report Series No. 24, Kingston, RI, April 1974, pp. 55-72.

Jones, D.B.; "Sloping Float Breakwater: Interim Data Summary," Technical Note No. 1568, U.S. Navy Civil Engineering Laboratory, Port Hueneme, CA, January 1980.

Davidson, D.D.; "Wave Transmission and Mooring Force Tests of Floating Breakwater, Oak Harbor, Washington," Technical Report H-71-3, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS, April 1971.

Laird, Alan D.K.; "Water Forces on Flexible Oscillating Cylinders," Journal of the Waterways and Harbors Division, ASCE, Vol. 88, No. WW3, August 1962, pp. 125-137.

Dean, R.G.; "Stream Function Representation of Nonlinear Ocean Waves," Journal of Geophysical Research, Vol. 70, No. 18, 1965.

Goda, Y.; "Irregular Wave Deformation in the Surf Zone," Coastal Engineering in Japan, Vol. 18, 1975, pp. 13-25.

Seelig, William N., and Ahrens, John P.; "The Elevation and Duration of Wave Crests," Unpublished Draft, U.S. Army Coastal Engineering Research Center, Fort Belvoir, VA, 2 January 1981.

Hall, Michael A.; "Laboratory Study of Breaking Wave Forces on Piles," Technical Memorandum No. 106, Department of the Army, Corps of Engineers, August 1958.

Ross, Culbertson W.; "Large-Scale Tests of Wave Forces on Piling (Preliminary Report)," Technical Memorandum No. 111, Beach Erosion Board, Department of the Army, Corps of Engineers, May 1959.

NAVFAC Documents. Department of Defense activities may obtain copies of Design Manuals and P-Publications from the Commanding Officer, Naval Publications and Forms Center, 5801 Tabor Avenue, Philadelphia, PA 19120. Department of Defense activities must use the Military Standard Requisitioning Procedure (MILSTRIP) using the stock control number obtained from NAVSUP Publication 2002.

Commercial organizations may procure Design Manuals and P-Publications from the Superintendent of Documents, U.S. Government Printing Office, Washington, DC 20420.

DM-3	Mechanical Engineering
DM-4	Electrical Engineering
DM-5	Civil Engineering
DM-7	Soil Mechanics, Foundations, and Earth Structures
DM-23	Port Control Offices
DM-25.1	Piers and Wharves
DM-25.2	Dockside Utilities for Ship Service
DM-25.3	Cargo Handling Facilities
DM-25.4	Seawalls, Bulkheads, and Quaywalls
DM-25.5	Ferry Terminals and Small Craft Berthing Facilities
DM-26.1	Harbors
DM-26.3	Coastal Sedimentation and Dredging
DM-26.5	Fleet Moorings
DM-26.6	Mooring Design Physical and Empirical Data
DM-38	Weight Handling Equipment and Service Craft

GLOSSARY

Added-Mass Coefficient. Empirically-derived hydrodynamic force coefficient, the "inertia" or "mass" coefficient.

Amplification Ratio. Correction for air-sea temperature difference.

Amplitude, Wave. The magnitude of the displacement of a wave from a mean value. An ocean wave has an amplitude equal to the vertical distance from stillwater level to wave crest. For a sinusoidal wave, amplitude is one-half the wave height.

Anchorage. A place to anchor.

Barge. An unpowered vessel used for transporting freight.

Basin. A naturally or artificially enclosed or nearly enclosed harbor area for small craft.

Bathymetry. The measurement of depths of water in oceans, seas, and lakes; also information derived from such measurements.

Bay. A recess in the shore or an inlet of a sea between two capes or headlands, not as large as a gulf but larger than a cove.

Beach Berm. A nearly horizontal part of the beach or backshore formed by the deposit of material by wave action. Some beaches have no berms, others have one or several.

Beach Nourishment. The process of replenishing a beach. It may be brought about naturally, by longshore transport, or artificially, by the delivery of materials dredged or excavated elsewhere.

Bent. A transverse group of piles together with a cap.

Breaker Height Index. Ratio, HÚb¿/H'Úo¿, of breaker height, HÚb¿, to unrefracted deepwater wave height, H'Úo¿.

Breakwater. A structure protecting a shore area, harbor, anchorage, or basin from waves.

Bulkhead. A structure or partition to retain or prevent sliding of the land. A secondary purpose is to protect the upland against damage from wave action.

Bypassing, Sand. Hydraulic or mechanical movement of sand from the accreting updrift side to the eroding downdrift side of an inlet or harbor entrance. The hydraulic movement may include natural movement as well as movement caused by man.

Capillary Wave. A wave whose velocity of propagation is controlled primarily by the surface tension of the liquid in which the wave is traveling. Water waves of length less than about 1 inch are considered capillary waves. Waves longer than 1 inch and shorter than 2 inches are in an indeterminate zone between capillary and Gravity Waves.

Celerity, Wave. Wave speed.

Channel. (1) A natural or artificial waterway of perceptible extent which either periodically or continuously contains moving water, or which forms a connecting link between two bodies of water. (2) The part of a body of water deep enough to be used for navigation through an area otherwise too shallow for navigation. (3) A large strait, as the English Channel. (4) The deepest part of a stream, bay, or strait through which the main volume or current of water flows.

Clapotis. The French equivalent for a type of standing wave. In American usage, clapotis is usually associated with the standing-wave phenomenon caused by the reflection of a nonbreaking wave train from a structure with a face that is vertical or nearly vertical. Full clapotis is one with 100-percent reflection of the incident wave; partial clapotis is one with less than 100 percent reflection. (See Standing Wave.)

Clay. Generally, fine-grained soils having particle diameters less than 0.002 millimeter.

Contour. A line on a map or chart representing points of equal elevation with relation to a datum. It is called an isobath when connecting points of equal depth below a datum.

Coral. (1) (Biology) Marine coelenterates (Madreporaria), solitary or colonial, which form a hard external covering of calcium compounds, or other materials. The corals which form large reefs are limited to warm, shallow waters, while those forming solitary, minute growths may be found in colder waters to great depths. (2) (Geology) The concretion of coral polyps, composed almost wholly of calcium carbonate, forming reefs, and tree-like and globular masses. May also include calcareous algae and other organisms producing calcareous secretions, such as bryozoans and hydrozoans.

Core (of Breakwater). Stone filler material, usually of small-sized stones as opposed to armor stone; applies principally to rubble-mound breakwaters.

Current. A flow of water.

Current, Tidal. The alternating horizontal movement of water associated with the rise and fall of the tide caused by the astronomical tide-producing forces.

Deep Water. Water so deep that surface waves are little affected by the ocean bottom. Generally, water deeper than one-half the surface wavelength is considered deep water.

Decay of Waves. The change waves undergo after they leave a generating area (Fetch) and pass through a calm, or region of lighter winds. In the process of decay, the significant wave height decreases and the significant wavelength increases.

Diffraction (of Water Waves). The phenomenon by which energy is transmitted laterally along a wave crest. When a part of a train of waves is interrupted by a barrier, such as a breakwater, the effect of diffraction is manifested by propagation of waves into the sheltered region within the barrier's geometric shadow.

Downdrift. The direction of predominant movement of littoral materials.

Eddy. A circular movement of water formed on the side of a main current. Eddies may be created at points where the main stream passes projecting obstructions or where two adjacent currents flow counter to each other.

Embankment. An artificial bank, such as a mound or dike, generally built to hold back water or to carry a roadway.

Erosion. The wearing away of land by the action of natural forces. On a beach, the carrying away of beach material by wave action, tidal currents, littoral currents, or by deflation.

Exp. Base of natural logarithm.

Fetch. The area in which Seas are generated by a wind having a rather constant direction and speed. Sometimes used synonymously with fetch length.

Filter Cloth. A synthetic textile with openings that allow water to escape, but which prevents the passage of soil particles.

Fillet. The accumulation of littoral material adjacent to a coastal structure such as a groin or a jetty.

Fines. The smaller particles of a granular material, such as silt and clay in sandy soils or sand in sandy gravel.

Floating Breakwater. Moored buoyant units for protecting harbors and shore areas from wave attack.

Fouling. The attachment and growth of marine plants and animals on surfaces of operational importance to man.

Freeboard. The additional height of a structure above design high water level to prevent overflow. Also, at a given time, the vertical distance between the water level and the top of the structure. On a ship, the distance from the waterline to main deck or gunwale.

Fully-Arisen Sea. The condition when the fetch length and duration are long enough for a given wind velocity to produce the highest waves possible. This steady wave state requires a minimum fetch and duration which can be related to the wind velocity at a specific height above the sea surface.

Gabion. A wire basket filled with stone or concrete rubble.

Geometric Shadow. In wave diffraction theory, the area outlined by drawing straight lines paralleling the direction of wave approach through the extremities of the protective structure. It differs from the actual protected area to the extent that the diffraction and refraction effects modify the wave pattern.

Gobi Block. A patented 8- by 8-inch concrete revetment block, 4 inches high, with a flat bottom, raised cobblelike top, and vertical holes to allow the escape of ground water, manufactured by Erco Systems of New Orleans.

Groin. A shore protection structure built (usually perpendicular to the shoreline) to trap littoral drift or retard erosion of the shore.

Groin Field. A series of groins acting together to protect a section of beach.

Group Velocity. The velocity of a wave group. In deep water, it is equal to one-half the velocity of the individual waves within the group.

Harbor. Any protected water area affording a place of safety for vessels.

Head, Breakwater. End or tip of breakwater.

Headland A high steep-faced promontory extending into the sea.

Hindcasting, Wave. The use of historic synoptic wind charts to calculate wave characteristics that probably occurred at some past time.

Hurricane. An intense tropical cyclone in which winds tend to spiral inward toward a core of low pressure, with maximum surface wind velocities that equal or exceed 75 miles per hour (65 knots) for several minutes or longer at some points. (Tropical storm is the term applied if maximum winds are less than 75 miles per hour.)

Inertial Coefficient. Empirically-derived hydrodynamic force coefficient, the "drag" coefficient.

Internal Waves. Waves that occur within a fluid whose density changes with depth, either abruptly at a sharp surface of discontinuity (an interface) or gradually. Their amplitude is greatest at the density discontinuity or, in the case of a gradual density change, somewhere in the interior of the fluid and not at the free upper surface where the surface waves have their maximum amplitude.

International Great Lakes Datum (IGLD). The common datum used in the Great Lakes area based on mean water level in the St. Lawrence River at Father Point, Quebec, and established in 1955.

Intertidal Zone. The land area that is alternately inundated and uncovered with the tides, usually considered to extend from mean low water to extreme high tide.

Jetty. On open seacoasts, a structure extending into a body of water, and designed to prevent shoaling of a channel by littoral materials, and to direct and confine the stream or tidal flow. Jetties are built at the mouth of a river or tidal inlet to help deepen and stabilize a channel.

Lee. Shelter, or the part or side sheltered or turned away from the wind or waves.

Littoral Transport. The movement of littoral drift in the littoral zone by waves and currents. Includes movement parallel (longshore transport) and perpendicular (on-offshore transport) to the shore.

Longshore Transport. Transport of sedimentary material parallel to the shore.

Low Water Datum (LWD). An approximation to the plane of mean low water that has been adopted as a standard reference plane.

Mach Reflection. Wave reflection which occurs when an incident wave encounters a structure at an oblique angle of less than 45 deg.. The incident wave crest turns so that it intersects the structure at a right angle.

Mach-Stem Wave. The wave which propagates along the axis of a structure, with its crest perpendicular to the structure, following the encounter of an incident wave with a structure at an oblique angle of less than 45 deg..

Mean Lower Low Water (MLLW). The average height of the lower low waters over a 19-year period. For shorter periods of observations, corrections are applied to eliminate known variations and reduce the results to the equivalent of a mean 19-year value. Frequently abbreviated to lower low water. (See Mean Low Water (MLW).)

Mean Low Water (MLW). The average height of the low waters over a 19-year period. For shorter periods of observations, corrections are applied to eliminate known variations and reduce the results to the equivalent of a mean 19-year value. All low water heights are included in the average where the type of tide is either semidiurnal or mixed. Only lower low water heights are included in the average where the type of tide is diurnal. So determined, mean low water in the latter case is the same as mean lower low water.

Mean Sea Level (MSL). The average height of the surface of the sea for all stages of the tide over a 19-year period, usually determined from hourly height readings. Not necessarily equal to mean tide level.

Mud. A fluid-to-plastic mixture of finely divided particles of solid material and water.

Nearshore (Zone). In beach terminology an indefinite zone extending seaward from the shoreline well beyond the breaker zone. It defines the area of nearshore currents.

Orbit. In water waves, the path of a water particle affected by the wave motion. In deepwater waves, the orbit is nearly circular and in shallow-water waves the orbit is nearly elliptical. In general, the orbits are slightly open in the direction of wave motion giving rise to mass transport.

Orthogonal. On a wave-refraction diagram, a line drawn perpendicularly to the wave crests.

Overtopping. Passing of water over the top of a structure as a result of wave runup or surge action.

Percolation. The process by which water flows through the interstices of a sediment. Specifically, in wave phenomena, the process by which wave action forces water through the interstices of the bottom sediment. Tends to reduce wave heights.

Pile, Sheet. A pile with a generally slender flat cross section to be driven into the ground or seabed and meshed or interlocked with like members to form a diaphragm, wall, or bulkhead.

Plunging Breaker. A wave breaking on a shore, over a reef, etc., where the crest curls over an air pocket; breaking is usually with a crash. Smooth splash-up usually follows.

Porosity. Ratio of volume of voids to total volume.

Recurrence Interval. The average number of years required for a particular wind or wave event to be exceeded.

Reflection (of Water Waves). A reflection of wave energy from natural or manmade barriers.

Reflection Coefficient. The ratio of the reflected wave height HÚr¿ to the incident wave height HÚi¿ is termed the reflection coefficient .

Refraction (of Water Waves). (1) The process by which the direction of a wave moving in shallow water at an angle to the contours is changed. The part of the wave advancing in shallower water moves more slowly than that part still advancing in deeper water, causing the wave crest to bend toward alignment with the underwater contours. (2) The bending of wave crests by currents.

Refraction Diagram. A drawing showing positions of wave crests and/or orthogonals in a given area for a specific deepwater wave period and direction.

Regular Reflection. Wave reflection which the angle of incidence equals the angle of reflection.

Revetment. A facing built to protect a scarp, embankment, or shore structure against erosion by wave action or currents, constructed of, for example, concrete or stone.

Reynolds Number. The dimensionless ratio of the intertial force to the viscous force in fluid motion,

$$R_e = \frac{LV}{\nu}$$

where L is a characteristic length, [nu] the kinematic viscosity, and V a characteristic velocity. The Reynolds number is of importance in the theory of hydrodynamic stability and the origin of turbulence.

Riprap. A layer, facing, or protective mound of stones randomly placed to prevent erosion, scour, or sloughing of a structure or embankment; also the stone so used.

Rubble-Mound Structure. A mound of randomly shaped and randomly placed stones protected with a cover layer of selected stones or specially shaped concrete armor units. (Armor units in the primary cover layer may be either placed in an orderly manner or dumped at random.)

Runup. The rush of water up a structure or beach on the breaking of a wave. The amount of runup is the vertical height above stillwater level that the rush of water reaches.

Salinity. The total amount in grams of solid material dissolved in 1 kilogram of seawater when all the carbonate has been converted to oxide, all the iodine and bromine have been replaced by chlorine, and all organic matter has been completely oxidized.

Scour. Removal of underwater material by waves and currents, especially at the base or toe of a shore structure.

Seas. Waves caused by wind at the place and time of observation.

Seawall. A structure separating land and water areas, primarily designed to prevent erosion and other damage due to wave action. (See also Bulkhead.)

Seiche. (1) A standing wave oscillation of an enclosed water body that continues, pendulum fashion, after the cessation of the originating force, which may have been either seismic or atmospheric. (2) An oscillation of a fluid body in response to a disturbing force having the same frequency as the natural frequency of the fluid system. Tides are now considered to be seiches induced primarily by the periodic forces caused by the sun and moon. (3) In the Great Lakes area, any sudden rise in the water of a harbor or a lake whether or not it is oscillatory. Although inaccurate in a strict sense, this usage is well-established in the Great Lakes area.

Shallow Water. (1) Commonly, water of such a depth that surface waves are noticeably affected by bottom topography. It is customary to consider water of depths less than one-half the surface wavelength as shallow water. (2) More strictly, in hydrodynamics with regard to progressive gravity waves, water in which the depth is less than 1/25 the wavelength.

Sheet Pile. See Pile, Sheet.

Shoal (verb). (1) To become shallow gradually. (2) To cause to become shallow. (3) To proceed from a greater to a lesser depth of water.

Shoaling Coefficient. The ratio of the height of a wave in water of any depth to its height in deep water, with the effects of refraction, friction, and percolation eliminated.

Significant Wave Height. The average height of the one-third highest waves of a given wave group. Note that the composition of the highest waves depends upon the extent to which the lower waves are considered. In wave-record analysis, the average height of the highest one-third of a selected number of waves, this number being determined by dividing the time of record by the significant period.

Soffit. The underside of a structural part, as of a beam, arch, or deck.

Standing Wave. A type of wave in which the surface of the water oscillates vertically between fixed points, called nodes, without progression. The points of maximum vertical rise and fall are called antinodes or loops. At the nodes, the underlying water particles exhibit no vertical motion, but maximum horizontal motion. At the antinodes, the underlying water particles have no horizontal motion but maximum vertical motion. They may be the result of two equal progressive wave trains traveling through each other in opposite directions. (See Clapotis.)

Still Water Level (SWL). The elevation that the surface of the water would assume if all wave action were absent.

Storm Surge. A rise above normal water level on the open coast due to the action of wind stress on the water surface. Storm surge resulting from a hurricane also includes that rise in level due to atmospheric pressure reduction as well as that due to wind stress.

Surf Zone. The area between the outermost breaker and the limit of wave uprush.

Swell. Wind-generated waves that have traveled out of their generating area. Swell characteristically exhibits a more regular and longer period, and has flatter crests than waves within their fetch.

Tidal Current. See Current, Tidal.

Tidal Inlet. (1) A natural inlet maintained by tidal flow. (2) Loosely, any inlet in which the tide ebbs and flows.

Tidal Range. The difference in height between consecutive high and low (or higher high and lower low) waters.

Tide. The periodic rising and falling of the water that results from the gravitational attraction of the moon and sun and other astronomical bodies acting upon the rotating earth. Although the accompanying horizontal movement of the water resulting from the same cause is also sometimes called Tide, it is preferable to designate the latter as Tidal Current, reserving the name Tide for the vertical movement.

Transitional Water. In regard to progressive gravity waves, water whose depth is less than one-half but more than 1/25 the wavelength. Often called Shallow Water.

Transmission Coefficient. The ratio of transmitted wave height to the incident wave height.

Trunk (of Breakwater). Main reach of breakwater.

Tsunami. A long-period wave caused by an underwater disturbance such as a volcanic eruption or earthquake. Commonly miscalled "tidal wave".

Turbidity. Quality or state of being turbid. Water which contains suspended matter which interferes with the passage of light through the water or in which visual depth is restricted is referred to as turbid.

Wave Crest. (1) The highest part of a wave. (2) That part of the wave above still water level.

Wave Height. The vertical distance between a crest and the preceding trough.

Wavelength. The horizontal distance between similar points on successive waves measured perpendicularly to the crest.

Wave Period. The time for a wave crest to traverse a distance equal to one wavelength. The time for two successive wave crests to pass a fixed point.

Wave, Reflected. That part of an incident wave that is returned seaward when a wave impinges on a steep beach, barrier, or other reflecting surface.

Wave Spectrum. In ocean wave studies, a graph, table, or mathematical equation showing the distribution of wave energy as a function of wave frequency. The spectrum may be based on observations or theoretical considerations. Several forms of graphical display are widely used.

Wave Transmission. When an incident wave strikes a structure, wave energy is transmitted through or over the structure.

Wave Trough. (1) The lowest part of a wave form between successive crests. (2) That part of a wave below still water level.

LIST OF SYMBOLS

Symbol	Definition
A	surface area
A	projected area of member
a	amplitude
a	breaking-wave dynamic moment reduction factor for low wall
B	gap spacing between breakwaters
B	crest width for rubble-mound structure
B	structure beam for floating breakwater
B'	imaginary equivalent breakwater gap
b	spacing between wave orthogonals
b	breakwater crest width
b	height above the bottom of rubble base (for wall built on rubble base) or of gap (for wave baffle)
b'	overtopped wall height above trough
C	wave celerity; phase velocity
CÚD¿	drag coefficient (dimensionless)
CÚM¿	inertial, or added mass, coefficient (dimensionless)
CÚg¿	group velocity
CÚt¿	windspeed conversion factor (where t = desired duration or given duration)
D	water depth one wavelength seaward (in front) of wall
D	- pile diameter - beam diameter
DÚi¿	respective pile diameter
d	water depth from still water level (SWL)
d/H'Úo¿	relative depth where toe of structure slope is at dÚs¿ = 0
d/L	relative depth (dimensionless)

Symbol	Definition
dÚb¿	depth of water at breaking
dÚb¿/HÚb¿	relative breaker depth
dÚs¿	- water depth from still water level at structure toe - water depth from still water level at toe of rubble foundation for wall built on rubble foundation
dÚs¿/H'Úo¿	relative depth
du/dt	instantaneous horizontal water-particle acceleration
dÚw¿	depth from still water level (SWL) at base of wall for wall built on rubble foundation
dÚi¿(etc.)	depth for fetch interval, FÚi¿
dÚ1¿	slope-protection depth; depth below still water level (SWL) of rubble-foundation crest
F	dynamic force per unit length of wall if wall is perpendicular to direction of wave approach
F	fetch length
F	structure freeboard
F'	reduced force on overtopped wall (nonbreaking wave)
FÚB¿	force per unit length of bracing
FÚD¿	linear drag force
FÚDSÚ[theta]¿¿	drag force corrected for nonlinear effects
FÚI¿	linear inertial force
FÚISÚ[theta¿¿	inertial force corrected for nonlinear effects
FÚT¿	- total force - total force on a pile subjected to nonbreaking waves
FÚTSÚ[theta]¿¿	total force on a pile at a given phase angle (corrected for nonlinear effects)
F"	reduced force on wall built on rubble base or force on baffle (nonbreaking wave)
FÚc¿	force on wall when crest is at wall

FÚc¿/wÚw¿ dÚs¿À2Ù dimensionless FÚc¿

Symbol	Definition
FÚm¿	dynamic component of force for breaking or broken wave
FÚmD¿	maximum drag force
FÚmDI¿	maximum (drag and inertial) force
FÚmI¿	maximum inertial force
F'Úm¿	corrected dynamic-impact force for overtopped wall
FÚn¿	component of F normal to wall
FÚs¿	hydrostatic component of force for breaking or broken wave
F'Ús¿	reduced hydrostatic moment for wall of low height
FÚt¿	force on wall when trough is at wall
FÚt¿/wÚw¿ dÚs¿À2Ù	dimensionless FÚt¿
FÚnet¿	net force on wall for nonbreaking wave
FÚu¿	uplift force
(F)ÚzÚi¿¿	force per unit length of bracing at an arbitrary distance above bottom, zÚi¿
FÚ[alpha]¿	reduced dynamic component of force for breaking or broken wave striking structure at oblique angle
FÚ[theta]¿	reduced horizontal dynamic component of force for breaking or broken wave striking nonvertical wall
FÚi¿	fetch for interval i
F'Ú1¿(etc.)	fetch length required to generate significant wave height, HÚs¿, if depth, d, had been dÚ2¿ in FÚ1¿, etc.
fÚD¿	drag force per unit length of pile
fÚI¿	inertial force per unit length of pile
g	gravitational acceleration (32.2 feet per secondÀ2Ù)
H	wave height measured between crest and trough; local wave height
H	average wave height; 0.626 HÚs¿ (not same as local wave height)

H/HÚb¿ relative wave height

Symbol	Definition
H/d	wave height relative to water depth
H/L	wave steepness
HÚb¿	- breaking-wave height - limiting height
HÚb¿/H'Úo¿	- relative breaker height - breaker height index
HÚi¿	incident wave height
HÚi¿/g TÀ2Ù	incident wave steepness
HÚm¿	Mach-stem wave height
HÚr¿	reflected wave height
HÚs¿	significant wave height; HÚ1/3; average height of highest one-third of waves for specified time period
HÚt¿	transmitted wave height
HÚt¿/HÚi¿	transmission coefficient (see KÚt¿)
HÚw¿	wave height at wall
HÚo¿	deepwater wave height
H'Úo¿	equivalent unrefracted deepwater wave height if wave unaffected by refraction and friction = HÚo¿ KÚR¿ KÚf¿ = H/KÚs¿
H'Úo¿/g TÀ2Ù	deepwater wave steepness
H'Úo¿/LÚo¿	deepwater wave steepness
HÚ1/3¿	average of highest one-third of all waves; HÚs¿ ; significant wave height
HÚ1¿	average of highest 1 percent of all waves for a given time period; 1.67 HÚs¿
HÚ10¿	average of highest 10 percent of all waves for a given time period; 1.27 HÚs¿
h	distance from SWL to bottom of structure (such as wall built on rubble base or wave baffle)
hÚc¿	height above bottom of core (of breakwater)

ÄÄÄ

Symbol	Definition
hÚc¿	height above still water level (SWL) of broken wave
hÚc¿/dÚs¿	relative core height
hÚo¿	height of clapotis orbit center above still water level (SWL)
hÚs¿	- height of structure - height of baffle structure from mudline to top of baffle
h'	broken-wave height above ground surface at structure toe shoreward of still water level SWL
KÚD¿	armor-unit stability coefficient
(KÚD¿)Úz¿	drag-force coefficient
(KÚDM¿)Úz=zÚi¿¿	linear maximum drag-force coefficient
(KÚI¿)Úz¿	inertial-force coefficient
(KÚIM¿)Úz=zÚi¿	linear maximum inertial-force coefficient
(KÚDMC¿ [phi]ÚD¿)	factor used in determining maximum drag force
KÚR¿	refraction coefficient
KÚf¿	decay coefficient
KÚr¿	reflection coefficient
KÚRR¿	stability coefficient for graded riprap
KÚs¿	shoaling coefficient; H/H'Úo¿
KÚsNL¿	nonlinear shoaling coefficient
K'	diffraction coefficient
KÚt¿	transmission coefficient; HÚt¿/HÚi¿
k	runup scale effect correction factor
kÚ[DELTA]¿	rock layer-thickness coefficient
L	wavelength
LÚD¿	wavelength in water depth D
LÚdÚs¿¿	wavelength in water depth dÚs¿

L/T	phase velocity, C

ÄÄ

Symbol	Definition
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ	ÄÄÄ
LÚo¿	deepwater wavelength
*l	structure slope length
M	moment about mudline
MÚB¿	moment about mudline corresponding to force per unit length of bracing, FÚB¿
MÚD¿	linear drag moment
MÚDSÚ[theta]¿¿	drag moment corrected for nonlinear effects
MÚI¿	linear inertial moment
MÚISÚ[theta]¿¿	inertial moment corrected for nonlinear effects
MÚTSÚ[theta]¿¿	total moment on a pile at a given phase angle
MÚc¿	moment about mudline of wall for nonbreaking wave when crest is at wall
MÚc¿/wÚw¿ dÚc¿À3Ù	dimensionless MÚc¿
MÚm¿	dynamic component of moment for breaking or broken wave
MÚmD¿	maximum drag moment
MÚmI¿	maximum inertial moment
MÚmDI¿	maximum (drag and inertial) moment
M'Úm¿	corrected dynamic moment about the mudline for overtopping breaking wave
MÚnet¿	net moment about mudline of wall for nonbreaking wave
MÚs¿	hydrostatic component of moment for breaking or broken wave
M'Ús¿	reduced hydrostatic force for wall of low height
MÚT¿	total moment
MÚt¿	moment when trough is at wall
MÚt¿/wÚw¿ dÚs¿À3Ù	dimensionless MÚt¿
M'	reduced moment about mudline for wall overtopped by nonbreaking wave

Symbol	Definition
M"ÚA¿	moment about mudline for wall built on rubble base (non-breaking wave)
M"ÚB¿	moment about base of wall for wall built on rubble base (nonbreaking wave)
m	bottom slope
NÚr¿	number of individual units in layer of interest
NÚr¿/A	placing density
n	- number of units comprising the layer of interest - number of armor units comprising the crest width
P	porosity
PÚc¿	pressure when clapotis crest is at wall
pÚm¿	maximum dynamic pressure by breaking and broken waves on vertical wall
pÚs¿	maximum hydrostatic pressure by broken wave
PÚt¿	pressure when clapotis trough is at wall
PÚ1¿	nonbreaking-wave pressure difference from still-water hydrostatic pressure as clapotis crest (or trough) passes
R	ratio of UÚW¿/UÚL¿
R	wave runup
R/H'Úo¿	relative runup
RÚe¿	Reynolds number
RÚT¿	amplification ratio
r	rough-slope runup correction factor
r	angle of reflection
r	layer thickness
rÚD¿	representative armor-stone diameter
rÚf¿	reduction factor for force on wall of height lower than clapotis crest

Symbol	Definition
rÚm¿	- reduction factor for moment on wall of height lower than clapotis crest - reduction factor for maximum dynamic component of force when breaking wave height is higher than wall height
S	distance to water free surface measured along the z-coordinate axis, a vertical axis with its origin at the bottom
SÚc¿	- depth from wave crest - distance of free surface measured from the bottom to the wave crest when the crest is at the pile
SÚr¿	specific gravity of armor unit based on the unit weight of water at the structure; wÚr¿/wÚw¿
SÚs¿	depth from soffit to bottom
SÚt¿	depth from wave trough
SÚ[theta]¿	distance of free surface measured from the bottom at an arbitrary wave-phase angle, [theta]
~SÚ[theta]¿	subscript SÚ[theta]¿ refers to values at an arbitrary wave-phase angle, [theta]
SÚ[theta]¿/d	relative free surface
T	wave period
TÚa¿	air temperature
TÚP¿	wave period associated with the highest peak of the wave spectrum ("peak spectral period")
TÚs¿	water temperature
t	- time - wind duration
UÚA¿	final adjusted windspeed used for hindcasting
UÚL¿	overland windspeed, adjusted for elevation and duration
UÚt¿=desired duration	windspeed at desired duration, adjusted for elevation and duration
UÚt¿= given duration	windspeed at given duration, adjusted for elevation

Symbol	Definition
U_W	overwater windspeed, adjusted for elevation and duration
U_z	windspeed at elevation z
U_{10}	windspeed at elevation of 10 meters
U'_A	windspeed corrected for nonconstant drag coefficient
u	instantaneous horizontal water-particle velocity
u_m	approximate maximum horizontal water-particle velocity
W	weight of individual armor unit or stone in layer of interest
W	parameter used in pile force and moment calculations
W_b	weight of back-slope armor unit
W_f	weight of front-slope armor unit
W_{50}	weight of 50-percent size of armor riprap gradation (50 percent of the material weighs W_{50} or more)
w	armor-stone weight
w	unit weight
w_r	unit weight of armor material (saturated surface dry)
w_w	unit weight of water (64 pounds per cubic foot for salt water)
x	coordinate axis in direction of wave propagation relative to wave crest
x_1	distance from still water level (SWL) to structure shoreward of still water level
x_2	distance from still water level (SWL) to limit of wave uprush
z	elevation of recorded wind
z	- vertical distance along a coordinate axis with its origin at still water level (SWL) (Section 1) - depth in terms of vertical distance along a coordinate axis with its origin at the bottom (Sections 5 and 7)

zÚB¿	distance of center of beam above bottom
zÚi¿	distance above bottom of point i

Symbol	Definition
zÚmD¿	lever arm; distance of point of application of drag force above bottom
zÚmDI¿	lever arm for drag and inertial force
zÚ1¿	depth from point 1
zÚ2¿	depth from point 2
[alpha]	angle between wave crest and bottom contour
[alpha]	angle between axis of wall and direction of wave approach
[alpha]	angle of incidence
[alpha]Úm¿	coefficient used in determination of maximum total moment on pile
[alpha]'	reflection-coefficient reduction factor (for multiple armor layers)
[alpha],[beta]	empirically derived constants which depend on structure type, water depth at structure toe, roughness of slope, number of armor layers, and breaking-wave height
[beta]	angle of beach slope with horizontal
[gamma]	unit weight of armor stone
[DELTA]	spacing between adjacent piles
[DELTA]Úx¿	spacing between piles in the direction of wave advance (that is, along the x-coordinate axis)
[DELTA]Úz¿	unit length of pile
[DELTA][theta]	phase difference between piles
[eta]	water-surface elevation at a given point relative to the still water level (SWL)
[eta]Úc¿	water-surface elevation at wave crest relative to still water level (SWL)
[eta]Úc¿/H	relative wave-crest elevation above still water level (SWL)
[theta]	angle of structure slope measured relative to the horizontal
[theta]	wave-phase angle

[theta]ÚFm¿ wave-phase angle for which total force is maximum

Symbol	Definition
[theta]ÚMm¿	wave-phase angle for which total moment is maximum
cot [theta]	cotangent of structure-slope angle, [theta]
tan [theta]	tangent of structure-slope angle, [theta]
[nu]	kinematic viscosity (approximately 1 x 10À -5Ù feetÀ2Ù per second for salt water)
[xi]	surf-similarity parameter
[pi]	constant = 3.14159
[rho]	density of water (2.0 slugs per foot for salt water)
([tau]ÚDM¿)Úz=zÚi¿¿	linear drag moment coefficient
([tau]ÚDMC¿[psi]ÚD¿)	factor used in determining drag moment
([tau]ÚD¿)Úz¿	drag-moment coefficient
([tau]ÚI¿)Úz¿	inertial-moment coefficient
([tau]ÚIM¿)Úz=zÚi¿¿	linear inertial moment coefficient
[phi]	angle of wave approach relative to breakwater
[phi]ÚD¿	drag-force correction factor for nonlinear velocity and acceleration fields
[phi]ÚDM¿	a nonlinear correction factor
[phi]ÚI¿	inertial-force correction factor for nonlinear velocity and acceleration fields
[phi]ÚIM¿	nonlinear inertial-force correction factor for maximum inertial force
[psi]Úm¿	coefficient used in determinAtion of maximum total force on piles
[psi]ÚD¿	drag-moment correction factor for nonlinear velocity and acceleration fields
[psi]ÚDM¿	a nonlinear correction factor
[psi]ÚI¿	inertial-moment correction factor for nonlinear velocity and acceleration fields
[psi]ÚIM¿	nonlinear inertial-moment correction factor for maximum inertial moment